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Proceedings of the Koblenz Symposium
September 1970

Hydrométrie

Actes du colloque de Coblence
Septembre 1970

Volume 1

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Preface

The International Hydrological Decade (IHD) 1965-74 was launched by the General Conference of Unesco at its thirteenth session to promote international co-operation in research and studies and the training of specialists and technicians in scientific hydrology. Its purpose is to enable all countries to make a fuller assessment of their water resources and a more rational use of them as man's demands for water constantly increase in face of developments in population, industry and agriculture. In 1972 National Committees for the Decade had been formed in 107 of Unesco's 130 Member States to carry out national activities within the programme of the Decade. The implementation of the programme is supervised by a Co-ordinating Council, composed of twenty-one Member States selected by the General Conference of Unesco, which studies proposals for developments of the programme, recommends projects of interest to all or a large number of countries, assists in the development of national and regional projects and co-ordinates international co-operation.

Promotion of collaboration in developing hydrological research techniques, diffusing hydrological data and planning hydrological installations is a major feature of the programme of the IHD which encompasses all aspects of hydrological studies and research. Hydrological investigations are encouraged at the national, regional and international level to strengthen and to improve the use of natural resources from a local and a global perspective. The programme provides a means for countries well advanced in hydrological research to exchange scientific views and for developing countries to benefit from this exchange of information in elaborating research projects and in implementing recent developments in the planning of hydrological installations.

As part of Unesco's contribution to the achievement of the objectives of the IHD the General Conference authorized the Director-General to collect, exchange and disseminate information concerning research on scientific hydrology and to facilitate contacts between research workers in this field. To this end Unesco has initiated two series of publications: *Studies and Reports in Hydrology* and *Technical Papers in Hydrology.*

The *Studies and Reports in Hydrology* series, in which the present volume is published, is aimed at recording data collected and the main results of hydrological studies undertaken within the framework of the Decade, as well as providing information on research techniques. Also included in the series will be proceedings of symposia. Thus, the series comprises the compilation of data, discussions of hydrological research techniques and findings, and guidance material for future scientific investigations. It is hoped that the volumes will furnish material of both practical and theoretical interest to hydrologists and governments participating in the IHD and respond to the needs of technicians and scientists concerned with problems of water in all countries.

A number of these volumes have been published jointly with the International Association of Hydrological Sciences and the World Meteorological Organization which have co-operated with Unesco in the implementation of several important projects of the IHD.

Préface

La Conférence générale de l'Unesco, à sa treizième session, a décidé de lancer, pour la période s'étendant de 1965 à 1974, la Décennie hydrologique internationale (DHI), entreprise mondiale visant à faire progresser la connaissance en matière d'hydrologie scientifique par un développement de la coopération internationale et par la formation de spécialistes et de techniciens. Au moment où l'expansion démographique et le développement industriel et agricole provoquent un accroissement constant des besoins en eau, la DHI permet à tous les pays de mieux évaluer leurs ressources hydrauliques et de les exploiter plus rationnellement.

Il existe actuellement dans 107 des 130 Etats membres de l'Unesco un comité national qui, pour tout ce qui a trait au programme de la Décennie, impulse les activités nationales et assure la participation de son pays aux entreprises régionales et internationales. L'exécution du programme de la DHI se fait sous la direction d'un Conseil de coordination composé de 21 Etats membres désignés par la Conférence générale de l'Unesco ; ce conseil étudie les propositions concernant le programme, recommande l'adoption de projets intéressant l'ensemble des pays ou un grand nombre d'entre eux, aide à la mise sur pied de projets nationaux et régionaux, et coordonne la coopération à l'échelon international.

Le programme de la DHI, qui porte sur tous les aspects des études et des recherches hydrologiques, vise essentiellement à développer la collaboration dans la mise au point des techniques de recherches, dans la diffusion des données hydrologiques, dans l'organisation des installations hydrologiques. Il encourage les enquêtes nationales, régionales et internationales tendant à accroître et à améliorer l'utilisation des ressources naturelles, dans une perspective locale et générale. Il permet aux pays avancés en matière de recherches hydrologiques d'échanger des informations ; aux pays en voie de développement, il offre la possibilité de profiter de ces échanges pour élaborer leurs projets de recherches et pour planifier leurs installations hydrologiques en tirant parti des acquisitions les plus récentes de l'hydrologie scientifique.

Pour permettre à l'Unesco de contribuer au succès de la DHI, la Conférence générale a autorisé le Directeur général à rassembler, à échanger et à diffuser des informations sur les recherches d'hydrologie scientifique et à faciliter les contacts entre les chercheurs dans ce domaine. A cette fin, l'Unesco fait paraître deux nouvelles collections de publications : "Etudes et rapports d'hydrologie" et "Notes techniques d'hydrologie".

La collection "Etudes et rapports d'hydrologie", dans laquelle est publié le présent ouvrage, a pour objet de présenter les données recueillies et les principaux résultats des études effectuées dans le cadre de la Décennie et de fournir des informations sur les techniques de recherche. On y trouve aussi les Actes de colloques réunis sur ce sujet. Cette collection publie donc des données, des techniques et des résultats de recherches ainsi qu'une documentation pour les travaux scientifiques futurs.

On espère que ces volumes apporteront aux hydrologues et aux gouvernements qui participent à la DHI des matériaux d'un intérêt tant pratique que théorique, et

qu'elle répondra aux besoins des techniciens et des hommes de science de tous pays qui s'occupent des problèmes de l'eau.

Certains de ces ouvrages sont publiés en coopération avec l'Association internationale des sciences hydrologiques ou l'Organisation météorologique mondiale dans le cadre de projets réalisés conjointement par ces organisations et l'Unesco.

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Foreword

Hydrometry has developed from a mere description of magnitudes of hydrological events to a modern science with sophisticated techniques for measuring phenomena in the water and on the surface of water, including nowadays the use of space technology.

The Co-ordinating Council for the International Hydrological Decade, recognizing the importance of hydrometry and the application of modern methods for evaluating and processing hydrological data, has developed various international programmes related to hydrometry. To give further impetus to the development of measuring techniques and to enable a broader application of the findings, Unesco convened an International Symposium on Hydrometry in Koblenz (Federal Republic of Germany) from 13 to 19 September 1970. This symposium was organized in co-operation with the World Meteorological Organization, the International Association of Hydrological Sciences and the National Committee for the IHD of the Federal Republic of Germany.

The technical preparation of the symposium was undertaken by an Organizing Committee and Secretariat under the leadership of Dr. J. Wallner, president of the Federal Institute for Hydrology, Koblenz. The symposium was attended by about 520 participants from fifty countries and four international organizations.

The scientific programme of the symposium covered the following main subjects:

Measurement of water stages and discharge;
Measurement of depth, temperature and water quality;
Measurement of solid matter transport;
Special techniques, including recording and teletransmission of data;
Evaluation of measured data.

All the work of the symposium was conducted in plenary session. The following scientists took the chair: Mr. S. Dumitrescu (Unesco), Mr. H. André (France), Mr. W. Friedrich (Federal Republic of Germany), Mr. R. Marcović (Yugoslavia), Mr. K. Subramanya (India), Mr. E. Walser (Switzerland), Mr. R. Zayc (Federal Republic of Germany), Mr. R. Carter (United States), Mr. R. Herschey (United Kingdom) and Professor L. J. Tison (International Association of Hydrological Sciences).

Eighty-seven papers submitted by specialists from twenty-one countries were discussed within the framework of the programme outlined above; they are reproduced in these proceedings with minor abbreviations and editorial changes. They are preceded by abstracts in English, French, Russian and Spanish. Reports of discussions on individual papers are given after the relevant paper.

It is hoped that these proceedings, published jointly by Unesco, WMO and IAHS, will be of value in transmitting information on the most up to date theories and techniques concerning hydrometry.

Avant-propos

L'hydrométrie, qui à ses débuts se bornait à décrire l'ampleur des phénomènes hydrologiques, est devenue une science moderne utilisant des techniques perfectionnées pour mesurer les phénomènes qui se produisent dans l'eau et à la surface, y compris, aujourd'hui, la technologie spatiale.

Le Conseil de coordination de la Décennie hydrologique internationale, conscient de l'importance que présentent l'hydrométrie et l'application de méthodes modernes à l'évaluation et au traitement des données hydrologiques, a élaboré divers programmes internationaux relatifs à l'hydrométrie. Pour stimuler encore davantage le développement des techniques de mesure et pour permettre une application plus large des découvertes, l'Unesco a réuni un Colloque international sur l'hydrométrie, à Coblence (République fédérale d'Allemagne), du 13 au 19 septembre 1970. Ce colloque a été organisé avec le concours de l'Organisation météorologique mondiale, de l'Association internationale d'hydrologie scientifique et du Comité national pour la DHI de la République fédérale d'Allemagne.

La préparation technique du colloque a été assurée par un comité d'organisation et un secrétariat sous la direction du Dr. J. Wallner, président de l'Institut fédéral d'hydrologie de Coblence. Environ 520 participants de 50 pays et de 4 organisations internationales ont pris part au colloque.

Le programme scientifique du colloque portait sur les principales questions suivantes :

Mesure des niveaux d'eau et jaugeage du débit.
Mesure de la profondeur, de la température et de la qualité de l'eau.
Mesure des transports solides.
Techniques spéciales, y compris l'enregistrement et la télétransmission des mesures.
Evaluation des mesures obtenues.

Tous les travaux du colloque se sont déroulés en séance plénière. Les spécialistes ci-après ont pris la parole : M. S. Dumitrescu (Unesco), M. H. André (France), M. W. Friedrich (République fédérale d'Allemagne), M. R. Marcović (Yougoslavie), M. K. Subramanya (Inde), M. E. Walser (Suisse), M. R. Zayc (République fédérale d'Allemagne), M. R. Carter (Etats-Unis d'Amérique), M. R. Herschey (Royaume-Uni) et le professeur L. J. Tison (Association internationale d'hydrologie scientifique).

Quatre-vingt-sept communications présentées par des spécialistes de vingt et un pays ont été examinées dans le cadre du programme esquissé ci-dessus. Elles sont reproduites dans les présents Actes, parfois légèrement abrégées ou avec quelques modifications de forme ; elles sont en outre précédées d'un résumé en anglais, en français, en espagnol et en russe. Les comptes rendus des débats sur une communication particulière suivent immédiatement le texte de celle-ci.

On espère que ces Actes, publiés conjointement par l'Unesco, l'OMM et l'AIHS, contribueront à la diffusion de l'information sur les théories et les techniques les plus modernes concernant l'hydrométrie.

I Measurement of water stages

Mesure des niveaux d'eau

The analysis of float and hydrostatic level gauges and the choice of optimal values of their basic elements

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Abstract. Optimum conditions for the operation of float water-level recorders are attained when the equilibrium of the system is disturbed only within strict limits during the rise or fall in the water level.

The method used in the design of the system makes it possible to select a given accuracy and a specified range of measurements at the basic design stage.

Analysis of the operation of hydrostatic level gauges shows that their net error lies within 1 cm of water level. Estimation of the allowable error permits a choice of the value of the quantization and the assessment of the principal elements in the design of the instrument for telemetry.

ANALYSE DES LIMNIMETRES A FLOTTEUR ET LIMNIMETRES HYDROSTATIQUES. CHOIX DES VALEURS OPTIMALES DE LEURS ELEMENTS PRINCIPAUX

Résumé. Les conditions optimales pour l'utilisation des limnimètres à flotteur sont atteintes lorsque le système est en déséquilibre dans les limites strictes pendant la montée et la baisse de niveau de l'eau. Les méthodes de calcul du système assurent le choix des éléments principaux constructifs du système avec une précision et une amplitude de mesure données.

L'analyse des limnimètres hydrostatiques témoigne que leur erreur totale ne dépasse pas 1 cm en hauteur d'eau. La détermination de cette erreur permet de choisir un pas optimal de la quantification du convertisseur et de calculer les éléments principaux de la construction d'appareils à transmission d'informations à distance.

ANÁLISIS DE MEDIDORES DE NIVEL HIDROSTÁTICOS Y DE FLOTADOR, Y ELECCIÓN DE VALORES ÓPTIMOS PARA SUS ELEMENTOS BÁSICOS

Resumen. Se alcanzan condiciones óptimas para el funcionamiento de los medidores de nivel de flotador en el caso de que el equilibrio del sistema se perturbe dentro de límites estrictos, en la elevación o descenso del nivel de agua.

El método para el cálculo del sistema permite la elección de sus elementos básicos de diseño para la precisión necesaria y para el tipo de mediciones a realizar.

El análisis del funcionamiento de medidores de nivel hidrostático muestra que su error neto es del orden de 1 cm.

La estimación del error permite elegir el paso de cuantificación del convertidor, y calcular los elementos principales del instrumento para la transmisión telemétrica de la información.

АНАЛИЗ ПОПЛАВКОВЫХ И ГИДРОСТАТИЧЕСКИХ УРОВНЕМЕРОВ И ВЫБОР ОПТИМАЛЬНЫХ ЗНАЧЕНИЙ ИХ ОСНОВНЫХ ЭЛЕМЕИТОВ

Резюме. Теория поплавковых уровнемеров при рассмотрении системы — поплавок поплавковое колесо, груз противовеса и др. — показывает, что оптимальная высота рабочей части поплавка в идеальном случае должна равняться сумме максимальных погрешностей в измерении уровня воды. Точность измерения уровня воды зависит от приращения подъемной силы поплавка при заданной амплитуде измерения. Сила тяжести груза противо-

веса оптимальна, когда она численно равна максимальному абсолютному значению приращения подъемной силы поплавка. Оптимальные условия работы прибора достигаются в случае, когда нарушается равновесие системы в строго определенных пределах при подъеме и спаде уровня воды. Методика расчета системы позволяет выбрать ее основные конструктивные элементы при заданной точности и амплитуде измерения.

Анализ гидростатических уровнемеров, в том числе телеизмерительных, показывает, что их суммарная методическая погрешность, связанная с влиянием изменчивости окружающей среды, находится в пределах 1-2 см уровня воды. Указанная методическая погрешность (без учета компенсации) позволяет выбрать шаг квантования пребразователя и рассчитать основные элементы конструкции прибора при дистанционной передаче информации.

The static régime of a float gauge is characterized by the equilibrium for the system (Fig. 1):

$$F_2 = F_1 - F_3 \tag{1}$$

where F_1 is the force of gravity acting on the float, F_2 is the force of gravity acting on the counterweight, F_3 is the carrying capacity of the float immersed to a depth X.

When the water surface is not enclosed then, according to Archimedes' principle:

$$F_3 = \gamma_w \cdot \int_0^x S(x) \cdot \mathrm{d}x$$

where $S(x)$ is the cross-sectional area of the float and γ_w is the specific weight of the surface water layer. For a float of constant cross-section

$$F_3 = \gamma_w \cdot S \cdot x,$$

where x is the depth to which the float is immersed. Since $S = \frac{\pi}{4} \cdot d^2$ at the depth x, the carrying capacity is

$$F_3 = \frac{\pi \cdot d^2 \cdot x \cdot \gamma_w}{4},$$

where d is the diameter of the cylindrical part of the float.

In a dynamic régime [ref. 1] the float follows the water level when equilibrium (1) is disturbed. For a fall in water level

$$F_2 < (F_1 - F_3) + \Delta F_3 \tag{2}$$

and for a rise in water level

$$F_2 > (F_1 - F_3) - \Delta F_3 \tag{3}$$

where ΔF_3 is the increment of the carrying capacity of the float. It is important that these inequalities should occur at the extremes of the measurement range. If Δh is the total increment of the depth of float immersion, then

$$\Delta F_3 = \frac{\pi \cdot d^2 \cdot \Delta h \cdot \gamma_w}{4} \tag{4}$$

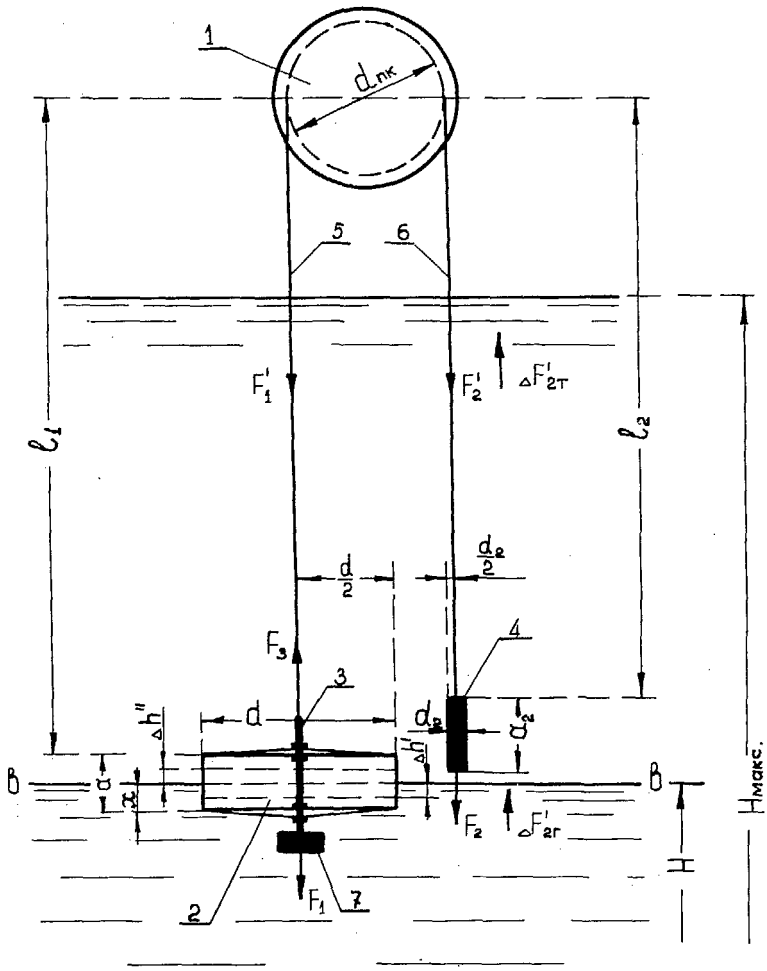


Figure 1. *Diagram of float level gauge*

1 – float wheel
2 – float
3 – fittings
4 – counterweight
5, 6 – cable
7 – additional weight.

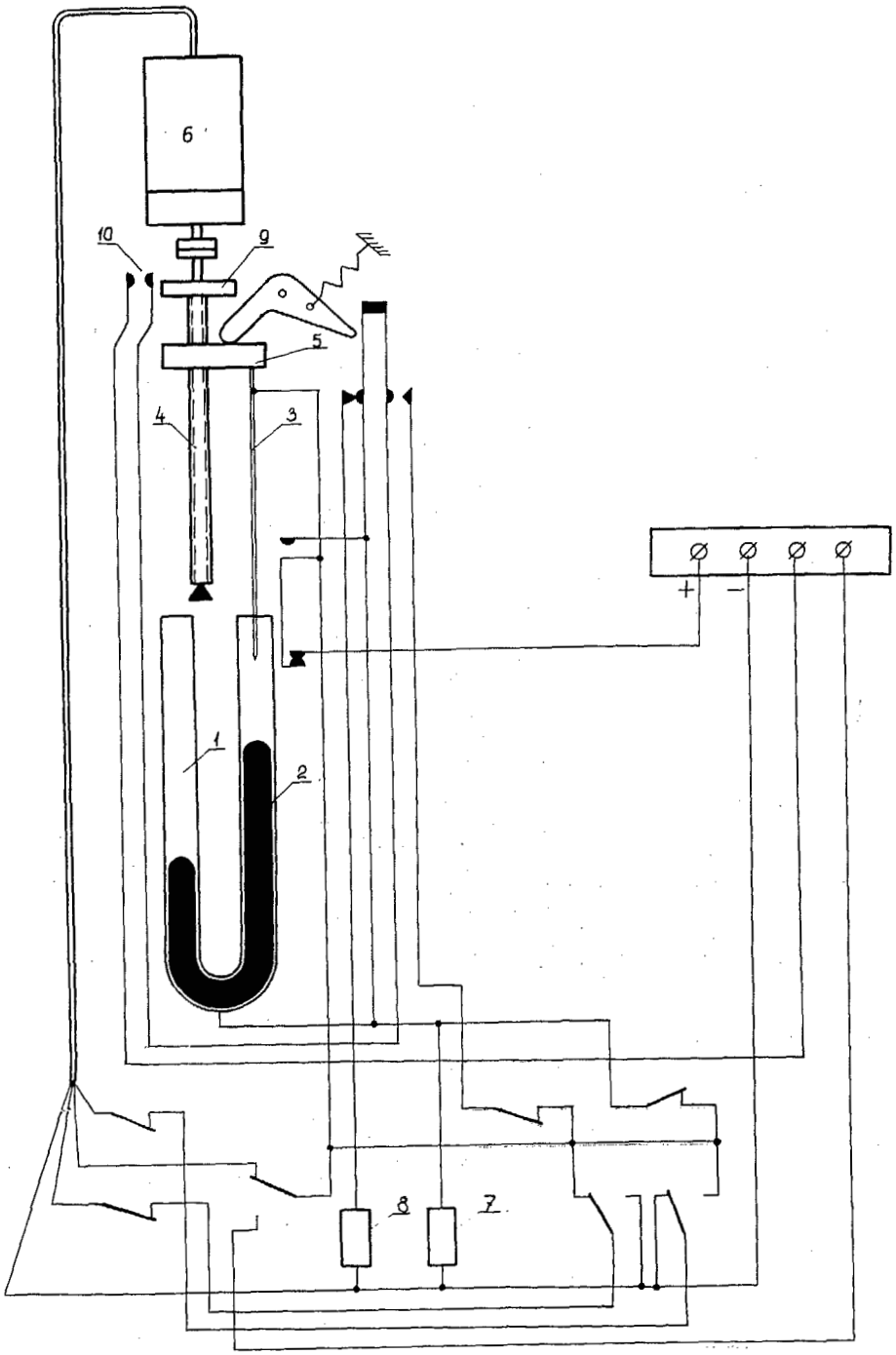


Figure 2. *Electrical scheme of hydrostatic level gauge.*

where $\Delta h = \Delta h' + \Delta h''$; and $\Delta h'$ and $\Delta h''$ are increments of the depth of float immersion during the fall and rise of water level respectively.

From inequalities (2) and (3) it follows that the float starts moving downwards when the water level is $\Delta h'$ lower than the horizon $b - b$. During the rise in level, F_3 becomes less by ΔF_3. The equilibrium is disturbed when the water level is $\Delta h''$ higher than the horizon $b - b$.

Inequalities (2) and (3) show that the force of gravity on the float (within the limits $F_2 = (F_1 - F_3) \pm \Delta F_3$) is automatically regulated by the carrying capacity of the float. It is evident that the change in the depth of float immersion shows the total error of the system. The increment of depth of float immersion is not constant and depends on many factors. From (4) it appears that

$$\Delta h = \frac{4 \cdot \Delta F_3}{\pi \cdot d^2 \cdot \gamma_w}$$

i.e. in order to diminish Δh, it is necessary either to diminish ΔF_3 or to increase the float diameter, which is more effective since the denominator includes the square of the diameter.

In general form:

$$\Delta h = f(F_1', F_2', \Delta F_2, F_{fr}, \gamma_w, \Delta l) \tag{5}$$

where F_1', F_2' are the forces of gravity acting on the left and right sides of the cable; ΔF_2 is the carrying capacity of the immersed part of the cable and the counterweight; F_{fr} is the frictional force acting on the instrument; Δl is the linear thermal expansion of the left part of the cable (l_1). Among the factors affecting the accuracy of measurement, F_1', F_2', ΔF_2 and to a certain extent F_{fr} are related to water level fluctuations, while γ_w and Δl depend on water and air temperature.

Where there is a fall in the water level (H) when $F_2' \to 0$ and $F_1' \to F_{1_{\max}}'$, the total acting force is

$$F_{3H}\downarrow = F_2' - F_1' + F_{fr} - \Delta F_2.$$

For a rise in water level, when $F_1' \to 0$ while $F_2' \to F_{2_{\max}}'$,

$$\Delta F_{3H}\uparrow = F_2' - F_{fr} - \Delta F_2 - F_1'$$

or

$$\Delta F_{3H}\uparrow = F_2' - F_1' - \Delta F_2 \pm F_{fr}.$$

Finally,

$$\Delta F_{3H} = \frac{\pi \cdot \gamma_w}{4} \cdot [d^2 (\Delta h_2' - \Delta h_1' \pm \Delta h_{fr}) - (d_2^2 \cdot a_2 + d_3^2 \cdot l_2)] \tag{6}$$

where $\Delta h_2'$, $\Delta h_1'$ and Δh_{fr} are the increments of float immersion due to the forces F_2', F_1' and F_{fr}; d_2 and a_2 are the diameter and the height of the counterweight; d_3 and l_2 are the diameter and the length of the submerged part of the cable.

The total error is

$$\Delta h_H = \Delta h_2' - \Delta h_1' - \Delta h_{2C} - \Delta h_{2T} \pm \Delta h_{\text{fr}}$$

where Δh_{2C} and Δh_{2T} are the errors due to the lifting forces of the counterweight and of the cable, respectively.

As is evident from equation 6 the float system has an error asymmetry.

Errors due to the changes of water density (ΔC) and air temperature (Δl) always have opposite signs. Therefore the total error is $\Delta h_m = \pm \Delta C \mp \Delta l$. The changes of ΔC and Δl occur due to daily and seasonal water and air temperature fluctuations. The resultant error value for the significant factors is

$$\Delta h = \sqrt{\Delta h_H^2 + \Delta h_m^2} \qquad \text{or}$$

$$\Delta h = \sqrt{\left(\frac{4 \cdot \Delta F_{3H}}{\pi \cdot d^2 \cdot \gamma_w}\right)^2 + (\pm \Delta l \mp \Delta C)^2}. \tag{7}$$

Here it is supposed that Δh_H and Δh_m appear independently of each other.

The value ΔC might be neglected in most cases, since under actual conditions it is less than 0.01 cm.

The design of the water level recorder mechansism depends mainly on the determination of Δh from equation 7.

Some basic conclusions may be reached on the design of a float-operated recorder:

1. The optimum height of the float's *cylindrical part (a)* should be a little greater or, ideally, equal to the maximum error amplitude, i.e. $\partial = \Delta h_{max}$;
2. The accuracy of measurement of the float gauge depends mainly on the increment of the float's carrying capacity ΔF_3 at given values of d_{nk} and H_{max} ;
3. The force of gravity on the counterweight F_2 is at an optimum when it is numerically equal to the maximum increment of the float's carrying capacity, i.e. $F_2 = \Delta F_{3max}$, provided that conditions (2) and (3) are observed;
4. The asymmetric error is characteristic of the float gauge, the range of the error depending on the choice (design) of the basic elements of the instrument.

Hydrostatic level gauges may be divided into three main groups: (a) combined or autonomic instruments; (b) instruments with a pneumatic or hydraulic connexion to the channel; (c) monitoring instruments with an electrical connexion. Hydrostatic level gauges with electrically operated communication lines seem to possess the greatest advantages for water stage observations, namely that they can be automated simply.

If the pipe (1) of the mercury manometer is affected by the hydrostatic water pressure P_w, while the pipe of the measuring unit (2) is affected by the atmospheric pressure P_a (Fig. 2), the equilibrium is

$$P_w - P_a = h_m \cdot \gamma_1 \quad \text{or} \quad h_m = \frac{P_w - P_a}{\gamma_1} \tag{8}$$

where h_m is the difference in the height of the liquid (mercury) columns in the manometer pipe, γ_1 is the specific weight of the liquid in the manometer. $\gamma_1 = \rho_1 g$, where ρ_1 is the density of the liquid in the manometer and g is gravitational acceleration. Thus, in order to measure water level *(H)* relative to the reference datum, it is sufficient to measure the difference in pressure in the manometer.

While calibrating the level gauge it is sufficient to establish the relation $h_m = f(H)$. Since equation 8 is linear, $h_m = f(H)$ is linear also.

The static characteristic of the level gauge is easy to determine. If the water level equals H and the specific weight of the liquid γ_w, the hydrostatic water pressure is $P_w = H \cdot \gamma_w$ and the height of the mercury column in the manometer is

$$h_m = \frac{P_w}{\gamma_1} \quad \text{or} \quad h_m = H \cdot \frac{\gamma_w}{\gamma_1} \,. \tag{9}$$

From equation 9 it is clear that the change in the specific weight of water and mercury due to temperature variations should be taken into consideration.

If a U-shaped manometer (Fig. 2) is placed in a bell of a cylindrical or conic shape and the pipe (1) is connected to the pressure receiver of the instrument, while the pipe of the measuring unit (2) is connected to the atmosphere, then the changes in the mercury column in the manometer due to the hydrostatic pressure of the water might be transformed into a digital impulse code. The cylindrical pipe in which the measuring unit of the instrument is placed is insulated from the pressure receiver. The pressure receiver is connected to the manometer by means of a brass pipe of small diameter, approximately 10 cm long.

The systematic errors in the instrument depend mainly on the temperature variations of its surroundings. These errors occur because of the thermal expansion in the junctions of the instrument ξ, the change of water density λ and the rise of water level in the pressure receiver (2). The analysis of these errors shows that, irrespective of temperature fluctuations, the resultant error

$$Z = (\xi + \lambda) - \epsilon \tag{10}$$

is almost completely compensated.

Thus, for instance, in the case of a range of measurement of water level from 0 to 10 m and temperature fluctuations from 0° to 25°C for fresh water, the average value $Z \leqslant 1$ cm, provided the diameters of manometer pipes 1 and 2 are equal. Experimental investigations corroborated these theoretical conclusions [2].

In the case of the electrically operated instrument, the principle used in the transformation of the linear movement of mercury into a digital impulse code is clearly shown. It consists of a pointer (3), a micrometer (4) with a sliding nut (5), reversible micromotor (6), two relays (7) and switching-over and blocking contacts (8).

In the case of the remote control of the instrument (manual or automatic) the pointer is lowered from its bench-mark position until it touches the mercury, then the motor is put into reverse by the relay and this raises the pointer to its initial position and thus the electric current to the motor is interrupted. When the pointer is lowered, a triple-edged interrupter sends a series of impulses to the communication line proportional to the water level. The unit of sampling (quantization) is close to 1 cm of water level. The discrepancy between the sampling unit and the actual value of the level is taken into account by the calibration of the instrument.

As experience of gauge operation in natural conditions shows, the total error P of the system (including the errors of conversion and recording) does not exceed 1 cm of water level, irrespective of the measurement range. The error is estimated from the formula

$$P = \sqrt{[(\xi + \lambda) - \epsilon]^2 + b_1^2} \tag{11}$$

where b_1 is the height of water level variation, corresponding to the number of unrecorded impulses.

The estimation of the error Z makes it possible to choose the value of the impulse and to determine the principal elements of instrument design. The distance of

telemetric transmission of the information depends on the voltage drop in the communication line and is practically unlimited. The instrument is fed by a source of constant current with voltage from 22 to 30 V, irrespective of the voltage drop in the line.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Dymaksian, A. M. 1967. Tochnost ismereniya i metodika rascheta poplavkovoy sistemy urovnemera [Measurement accuracy and computation technique for float system of a level gauge]. *Trudy Gos. Gidrologicheskogo Instituta,* vyp. 249.
2. Dymaksian, A. M. 1957. *Novye teleismeritelnye gidrometeorologicheskiya pribory* [New telemetric hydrometeorological instruments]. Leningrad, Gidrometeoisdat.

Digital recording of water levels with the aid of acoustics and its application to hydrological pumping tests

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Abstract. In pumping tests, the desired data can frequently be obtained only at considerable expense and effort.

In most cases, manual recording involves high labour costs and yields few and insufficiently accurate measurements. Conventional automatic devices are often subject to mechanical disturbances and are often unable to follow quick changes in stage, due to the mechanical follower arrangement of floats and probes. As a result, the cost of instrumentation rises almost linearly with the number of measuring points. The immense number of measured values resulting from a pumping test is presented in the form of lists or analogue records; detailed evaluation requires an extremely long time.

To overcome these difficulties by modern techniques, the following measuring method was chosen: ultrasonic vibrators acting as transmitters and receivers are suspended motionless in the water at the ground-water observation points. The transit time of an ultrasonic pulse travelling from the vibrator to the water surface and back is measured with the aid of an extremely accurate microchronometer. Multiplication of this transit time by the sound velocity gives the distance between vibrator and water surface. The resolving power is about 1 mm. The distances between vibrator and water surface are measured in sequence and recorded (as transit time), together with date and hour of the measurement and the number of the measuring point, on punched tape. From the punched tape, the data are transferred to a magnetic disc where they are available for direct use by the computer. The drawdown curves are graphically and numerically evaluated with the aid of various programmes. The resulting possibilities are indicated by means of several examples.

ENREGISTREMENT DIGITAL DES NIVEAUX DE L'EAU PAR DES METHODES ACOUSTIQUES ET APPLICATION A DES TESTS HYDROLOGIQUES DE POMPAGE

Résumé. Ce n'est qu'à grands frais qu'on peut obtenir les données de mesure désirables à l'aide d'essais de pompage.

L'enregistrement manuel fournit, dans la majorité des cas, des données trop peu nombreuses et trop inexactes et nécessite des frais de personnel trop élevés. Les appareils automatiques qui étaient en usage jusqu'ici se sont fréquemment trouvés en réparation et, souvent, ils ne sont pas capables de suivre les variations rapides des niveaux d'eau souterraine à cause du système mécanique de poursuite des flotteurs d'enregistrement ou des sondes. Dans le cas de ce mode opératoire technique, les dépenses pour les appareils augmentent à peu près de façon linéaire par rapport aux postes d'observation des niveaux d'eau souterraine qu'il faut saisir. Le nombre immense des valeurs enregistrées lors d'un essai de pompage se présente sous forme de listes ou enregistrements analogues ; une évaluation détaillée requiert un temps extraordinairement élevé.

Afin de surmonter ces difficultés et afin d'effectuer la mise en service rationnelle de moyens modernes, la méthode de mesure suivante a été choisie : des vibrateurs ultrasoniques utilisés comme postes émetteurs et récepteurs sont suspendus immobiles dans les postes d'observation des niveaux d'eau souterraine tout en étant plongés dans l'eau. La durée du parcours d'une impulsion ultrasonique allant depuis le vibrateur ultrasonique à la surface de l'eau et retour est mesurée à l'aide d'un appareil d'une précision extrême pour mesurer les petits intervalles de temps. Cette durée du parcours donne, après multiplication par la vitesse du son, la distance entre le vibrateur ultrasonique et la surface de l'eau. Le pouvoir résolvant s'élève à 1 mm. Ces distances entre vibrateur et surface de l'eau sont interrogées à l'aide d'une programmation appropriée et sont poinçonnées sur une bande perforée (sous forme de la durée du parcours) conjointement

avec la date et l'heure précises de la mesure ainsi qu'avec le numéro du poste d'observation. Les données sont reportées depuis la bande perforée sur une plaque de mise en mémoire magnétique ; puis elles sont disponibles à la lecture directe de l'ordinateur. Les courbes d'abaissement de l'eau souterraine sont évaluées du point de vue graphique et numérique à l'aide de programmations diverses. Les possibilités déduites de cette évaluation graphique et numérique sont montrées à l'aide de quelques exemples.

MEDIDOR AUTOMÁTICO DE NIVELES CON LA AYUDA DE APARATOS ACÚSTICOS Y SU APLICACIÓN A LOS ENSAYOS DE BOMBEO

Resumen. Las medidas necesarias en las pruebas de bombeo, se obtienen normalmente con dificultad y coste elevado.

En la mayoría de los casos, el registro manual da lugar a un número insuficiente de datos de escasa precisión y origina elevados costos de mano de obra. Los dispositivos automáticos convencionales han mostrado frecuentemente su propensión a la interferencia, y muchas veces son incapaces de seguir cambios de nivel que se produzcan con rapidez, debido al dispositivo de seguimiento mecánico de flotadores e indicadores. En consecuencia, los costos de equipo se elevan casi linealmente con el número de puntos de medida a cubrir. El gran número de medidas que resultan de una prueba de bombeo se presenta en forma de listas o resúmenes, exigiendo su evaluación detallada tiempos extremadamente largos.

Para superar estas dificultades y permitir la utilización de medios modernos, se eligió el siguiente método de medición: se introducen en el agua vibradores ultrasónicos, que actúan como transmisores y receptores sin movimiento en el agua, en los puntos básicos de observación. El tiempo de tránsito de un esfuerzo ultrasónico que se desplaza desde el vibrador a la superficie del agua y vuelve, se mide con ayuda de un microcronómetro extremadamente preciso. Al multiplicar este tiempo de tránsito por la velocidad del sonido, se obtiene la distancia entre el vibrador y la superficie del agua. El poder de apreciación viene a ser de 1 mm. Las distancias entre el vibrador y la superficie del agua se mide a intervalos, y se registran (como tiempo de tránsito) con la fecha y hora de su medición, y el número de puntos de medida, sobre cinta perforada. A partir de la cinta perforada, los datos se pasan a un disco magnético que puede introducirse directamente en un ordenador. Mediante diversos programas, se pueden evaluar gráfica y numéricamente las curvas de descenso de superficie. En varios ejemplos se indican las posibilidades de utilización del método.

ЦИФРОВАЯ РЕГИСТРАЦИЯ УРОВНЕЙ ВОДЫ С ПОМОЩЬЮ АКУСТИКИ И ЕЕ ПРИМЕНЕНИЕ В ОПЫТНЫХ ОТКАЧКАХ

Резюме. В опытах по откачкам требуемые данные измерений часто могут быть получены только при значительных усилиях и затратах.

Во многих случаях регистрация вручную дает недостаточное количество не вполне точной информации и вызывает высокую стоимость работы.

Общепринятые автоматические устройства часто оказываются подверженными интерференции и не способны следовать за быстрыми изменениями уровня воды из-за механического устройства, состоящего из поплавков и зондов. В результате стоимость оборудования возрастает почти пропорционально числу точек, в которых необходимо выполнить измерения. Огромное количество измеренных значений, получающихся в опытах по откачкам, представлены в виде списков или аналогичных записей; детальная оценка требует очень много времени.

Чтобы преодолеть эти трудности и сделать возможным использование современных средств, был выбран следующий метод измерений: ультразвуковые вибраторы, работающие как передатчики и приемники, неподвижно закрепляются в воде в точках, где проводятся наблюдения за грунтовыми водами. Время прохождения ультразвукового импульса от вибратора до поверхности воды и обратно измеряется с помощью очень точного микрохронометра. Умножение этого времени на скорость звука дает расстояние между вибратором и водной поверхностью. Разрешающая способность около 1 мм. Расстояния между вибратором и водной поверхностью считываются посредством соответствующих программ и регистрируются на перфоленте

(как время прохождения ультразвука) вместе с датой и часом измерений и количесвом измеренных точек. С перфоленты данные переносятся на магнитный диск, где они доступны для непосредственной обработки компьютером. Кривые понижения поверхности устанавливаются графическими и числовыми методами с помощью различеных программ. Возможные результаты показаны посредством нескольких примеров.

1 INTRODUCTION

It is frequently found that detailed measurements during hydrological pumping tests can be carried out only at very great expense. Manual recording may yield data which are too sporadic and inaccurate. On the other hand, much of the usual equipment for automatic recording which is on the market is relatively susceptible to mechanical disturbances and is not sufficiently adaptable in the event of abrupt change of water level. The cost of such equipment increases approximately linearly with the number of recording stations. Finally, a detailed interpretation of the great number of measured values which are accumulated in lists or on analogue recording charts necessitates a great expenditure of time.

To avoid these difficulties in the recording and evaluation of pumping tests, a method has been developed which makes use of modern electronic equipment and techniques. The travel time of ultrasonic pulses can be measured with an accuracy of 0.1 μs. This corresponds to an accuracy of about 1 mm in determining the distance of a vibrator from the water level in an observation well. The data may be transferred to a magnetic disc store where they are directly accessible to the digital computer. It is then possible to digest and interpret many more data than are usually available in order to characterize the drawdown around a pumping well as a function of time and position more accurately.

2 THE MEASURING DEVICE

2.1 Fundamentals

The method employed is based on the measurement of transit-time to and from the water surface of an ultrasonic pulse emitted by a vibrator suspended at a fixed depth below the water level.

The ultrasonic vibrator is used simultaneously as a pulse generator and as a pulse receiver. Measurement is effected by means of an echo-sounding device for ultrasonic pulses. A sonic velocity in the water of approximately 1,450 $m.s^{-1}$ gives a transit-time of approximately 1.4 μs per mm of distance. In order to achieve a resolution of 1 mm, an accuracy of 0.1 μs for the transit-time is required.

2.2 Selection of the ultrasonic vibrators

To reduce costs, it was decided not to use a special design for the vibrators. Instead, ultrasonic vibrators which are already in series production were relied upon. Several factors should be considered in choosing the frequency of the ultrasonic vibrator. It must be high enough for the rise-time of pulses to be less than 1.4 μs, equivalent to a transit-time of 1 mm at the desired resolution. The higher the frequency, the smaller

are the dimensions of the ultrasonic vibrators that can be supplied in series for equal ultrasonic power, and the smaller is the radiation angle of the vibrator. On the other hand, as the absorption of ultrasonic energy in water increases as the second power of the frequency, the lowest possible frequency ought to be chosen. Hence, after testing various ultrasonic vibrators, a frequency of 1 MHz was chosen, for which the rise-time of pulses is about 0.25 μs. At this frequency only piezoelectric transformers can be considered.

2.3 *Construction of the measuring probe*

A commercial ultrasonic vibrator with a resonance frequency of 1 MHz was chosen. The diameter of the ultrasonic vibrator is 22 mm, so that a probe of 45 mm diameter is possible for use in 2 inch (51 mm) tubes. The probes are approximately 450 mm long to prevent tilting in the tube and are freely suspended in the observation wells at some distance below the water level. The weight of the probe chosen is such that the cable is kept taut.

2.4 *Measuring apparatus*

A block diagram is shown in Fig. 1. The device is subdivided into analogue and digital parts. The analogue part is made up of the pulse generator, the amplifier for the reflected pulses and a scanner. The digital part contains a digital clock, the programming unit and the output unit for data, with a tape perforator as a recording device. The electronic counter for short-time measurement combines the two parts of the device. It transforms the analogue transit-time into a digital value.

2.4.1 *The pulse generator*

The pulse generator is a tube generator which, after being triggered, generates a 1 MHz highly damped sine wave; the damping is of approximately 20 db/period. The amplitude is approximately 500 V with the ultrasonic vibrator connected to a cable 15 m long.

2.4.2 *The amplifier for the reflected pulses*

The amplifier for the reflected pulses must have a high input sensitivity and, at the same time, an extreme input overdrive capacity, as it first receives the full voltage of the radiated pulse at the input and, only a few microseconds later, the reflected pulses of but a few microvolts or millivolts. It is constructed as a fully transistorized amplifier. The amplifier has a limiting stage in the input followed by impedance-transforming stages which increase the input resistance. The next stage is selected by means of a resonance circuit tuned to 1 MHz. The output of the amplifier is of extremely low resistance so as to choke-off damped oscillations; this is effected by means of a wide-band transformer, so that the output is free of d.c. voltage. After that the signal is rectified and passes through a filter network whose time constants have been chosen so that the output signal represents in practice the envelope curve of the amplified and rectified echo pulses. This output signal cannot be utilized immediately as an input to stop the counter, since it also includes energy derived from the directly radiated pulse as well as statistically distributed external and internal noise. Therefore a threshold circuit has been incorporated into the subsequent output amplifier to transmit only signals that exceed a preset amplitude. The direct signal is cut off by a grating circuit which opens an amplifying path only after a certain adjustable time interval after departure of the transmitted pulse and blocks

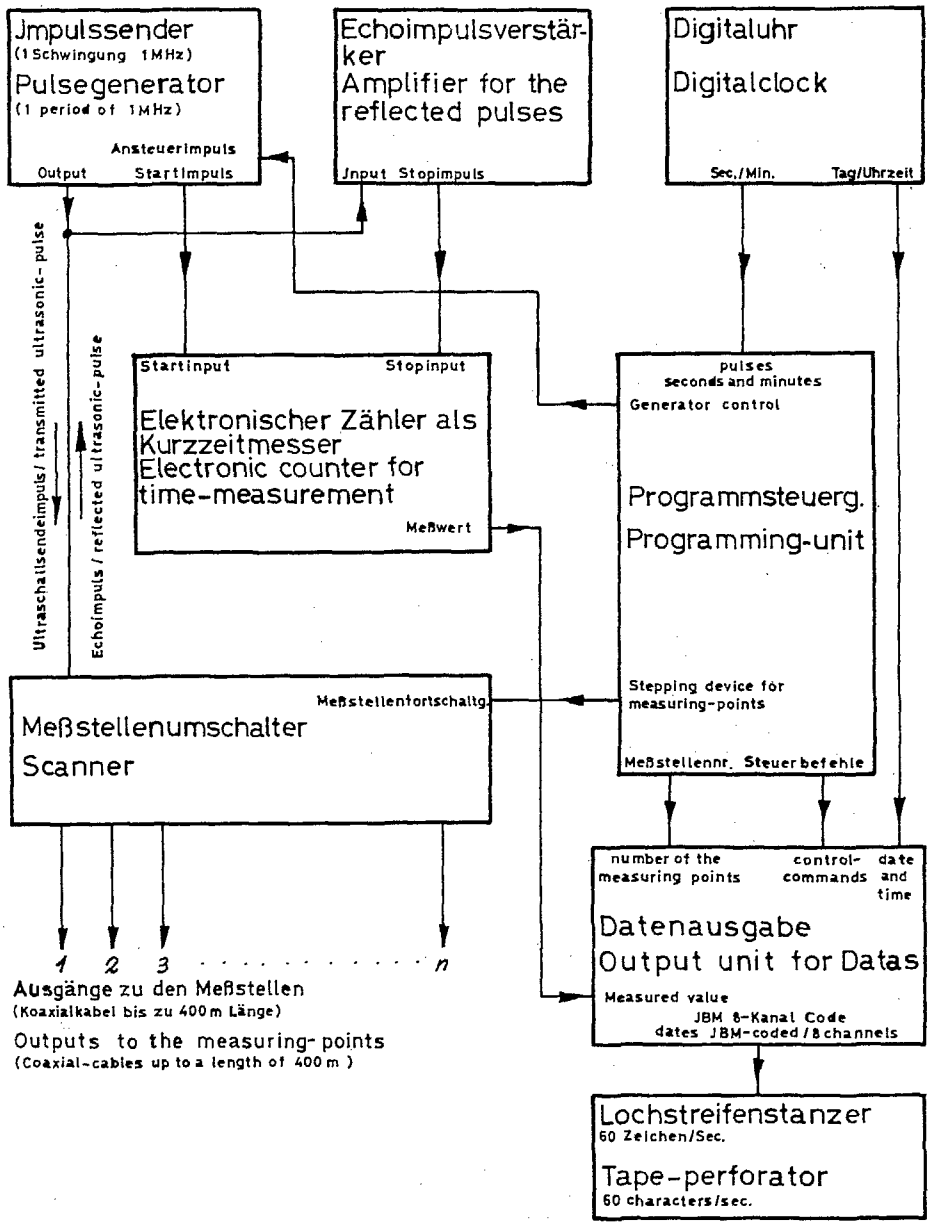


Figure 1. *Determination of the water level by measuring the transit time of a sonic pulse.*

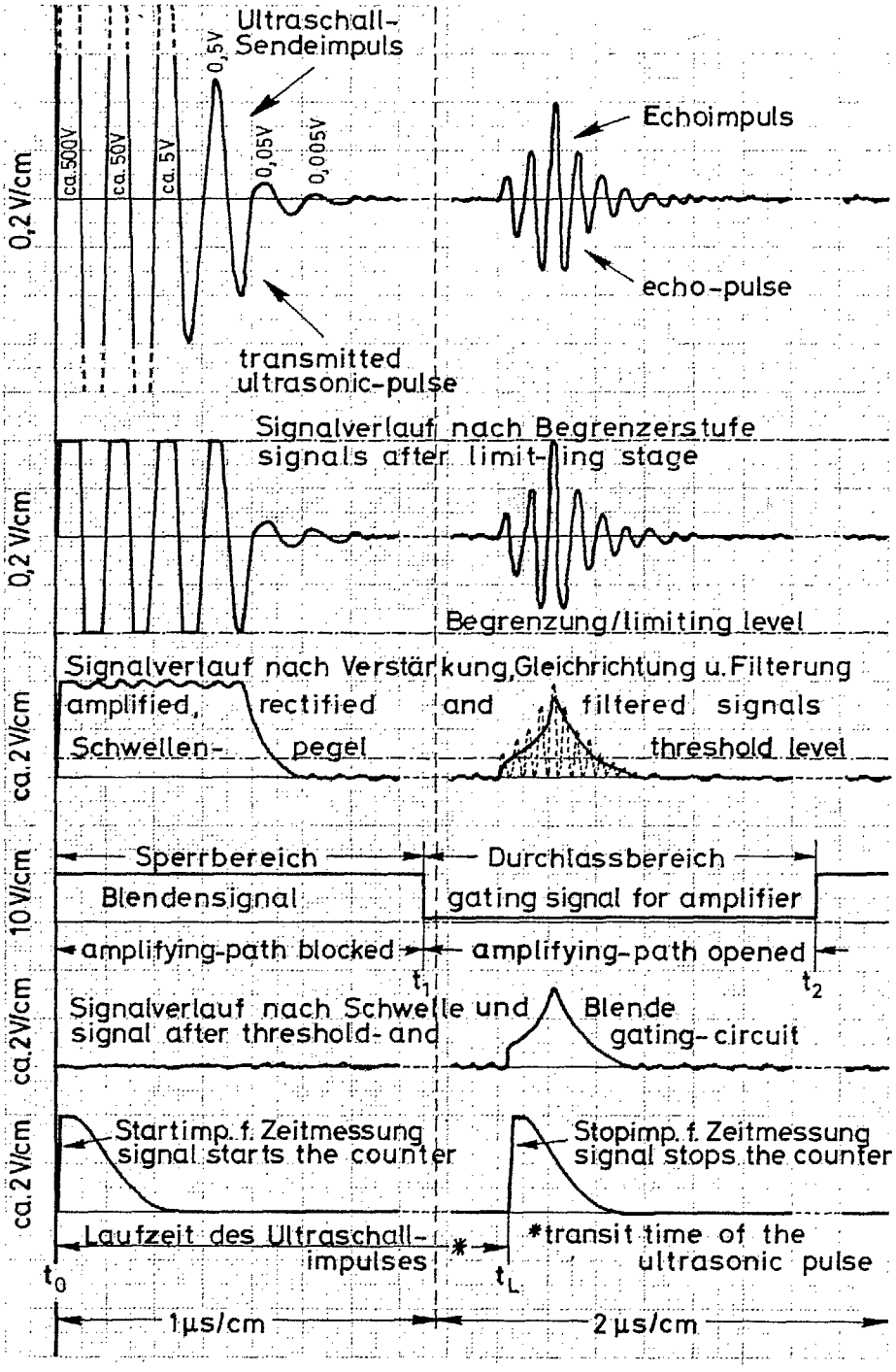


Figure 2. *Echo-pulses.*

it again after a certain time interval, longer than the maximum expected measuring time. In this way, only selected pulses are observed. The interdependent amplitudes and time interval conditions are shown in Fig. 2.

2.3.4 The device for time measurements

The device for time measurements is an electronic counter for a frequency of up to 30 MHz. It counts the time units (0.1 μs) from the opening of the gate by the starting pulse, which is derived from the leading edge of the direct pulse, until the gate is closed by the leading edge of the amplified echo pulse. It thereby measures the transit time of the ultrasonic pulse from the ultrasonic vibrator to and from the water level. The transit time is proportional to the distance between ultrasonic vibrator and water level. This measured value is stored, to be called off by the data output unit before the next measurement.

2.4.4 The scanner

The scanner connects this whole installation to the ultrasonic vibrator in the observation well. The switching is done by a special scanner which contains relays with a coaxial switch. Scanners of commercial quality are not entirely suitable for transmitting the radiated pulses.

2.4.5 The digital clock

The digital clock places the second and minute pulses at the disposal of the programming unit and makes possible the selection of the appropriate measuring day and the block time form the store.

2.4.6 The programming unit

The programming unit determines the chronological order of the measuring programme. It places the triggering pulse at the disposal of the ultrasonic pulse generator at the precise times required and provides sychronization with the appropriate measuring point.

As the water level changes quite rapidly in the immediate vicinity of a pumping well it is desirable to record more often than from more remote observation wells. Therefore a full cycle of measurements can be followed by a number of partial cycles covering only the wells in the vicinity of the pumping well.

The time interval between measuring cycles can be adjusted so that frequent measurements are taken at the beginning of the test, whereas less frequent measurements at larger time intervals are taken later. This entails a considerable reduction in the quantity of data obtained, while providing a great amount of information about the variations at the beginning of the test. Within the measuring cycles, measurements are carried out at the rate of one per second. In addition, the programming unit controls the output of data.

2.4.7 The output of data

The output of data is in IBM 8-channel-code on the punched tape in a bit-parallel fashion with serial progress for each instruction signal. At the beginning of each measuring cycle, the day, hour and minute is recorded; then the number of the measuring point and the measured values are punched successively, the latter up to five decimal places. The processed punched tape accordingly contains all the data

required and can subsequently be fed into a data-processing installation for further evaluation.

3 PRACTICAL CONSIDERATIONS

3.1 Deviation of the tubes from the vertical, length of cables and placing of vibrators

Difficulties will arise in the event of deviations of the observation well axis from the vertical. Tests have proved that measurements can be carried out with deviations of up to 10° with the vibrator 2 m below the water level. The echo amplitudes are sensibly diminished with increasing deviation from the vertical, but they are still adequate and within the above-mentioned limits.

The maximum distance between vibrator and water level, and the length of the cables between vibrator and measuring device, are limited by the impedance of the cables. For the cables used in these experiments and the particular pulse generator and echo-amplifier employed, the maximum distance between vibrator and water level is about 4 m, assuming that the length of the cable is less than 100 m. A cable length up to 40 m is practicable if the distance between vibrator and water-level is less than 1 m.

3.2 Measuring the actual sonic velocity

The distance in metres to the water surface, a, is given by the formula

$$a = 0.5 \cdot 10^{-6} \cdot t \cdot v$$

which necessitates precise determination of the sonic velocity v (m . s^{-1}). To achieve this, one measuring point is switched to a special probe. This probe contains a reflecting plate at an exact distance of 1 m from the vibrator. The measured transit-time $t(\mu s)$ for the distance of 1 m between vibrator and reflecting plate and back allows the calculation of the velocity by

$$v \ (\mathrm{m} \cdot \mathrm{s}^{-1}) = 2.10^6 \cdot (1/t \ [\mu s]) .$$

This sonic velocity, being representative for the water in the measuring area, is basic to the remainder of the calculations.

3.3 Errors of measurement

Sonic velocity in water depends upon temperature and salinity (Fig. 4). Changes of these will therefore affect the measuring accuracy. The possible deviation of the sonic velocity from the approximate value v = 1,450 m . s^{-1} can be evaluated.

As shown in Fig. 4, the sonic velocity increases by 4 m. s^{-1}/° C in the interval considered. This corresponds to an error F_s = 0.26 %/° C, i.e. 2.6 mm/° C for a measuring distance of 1 m. An increase in salinity of 10^3 ppm produces an increase of the sonic velocity of F_s = 0.09 %/10^3 ppm. Because the salinity in the ground water is about 50 to 100 ppm, changes in the salinity during the pumping test may be neglected. The effect of changes in temperature on the required accuracy must be considered, since changes of up to 1.5° C have been observed in pumping tests, corresponding to an error of 4 mm per metre of measured distance.

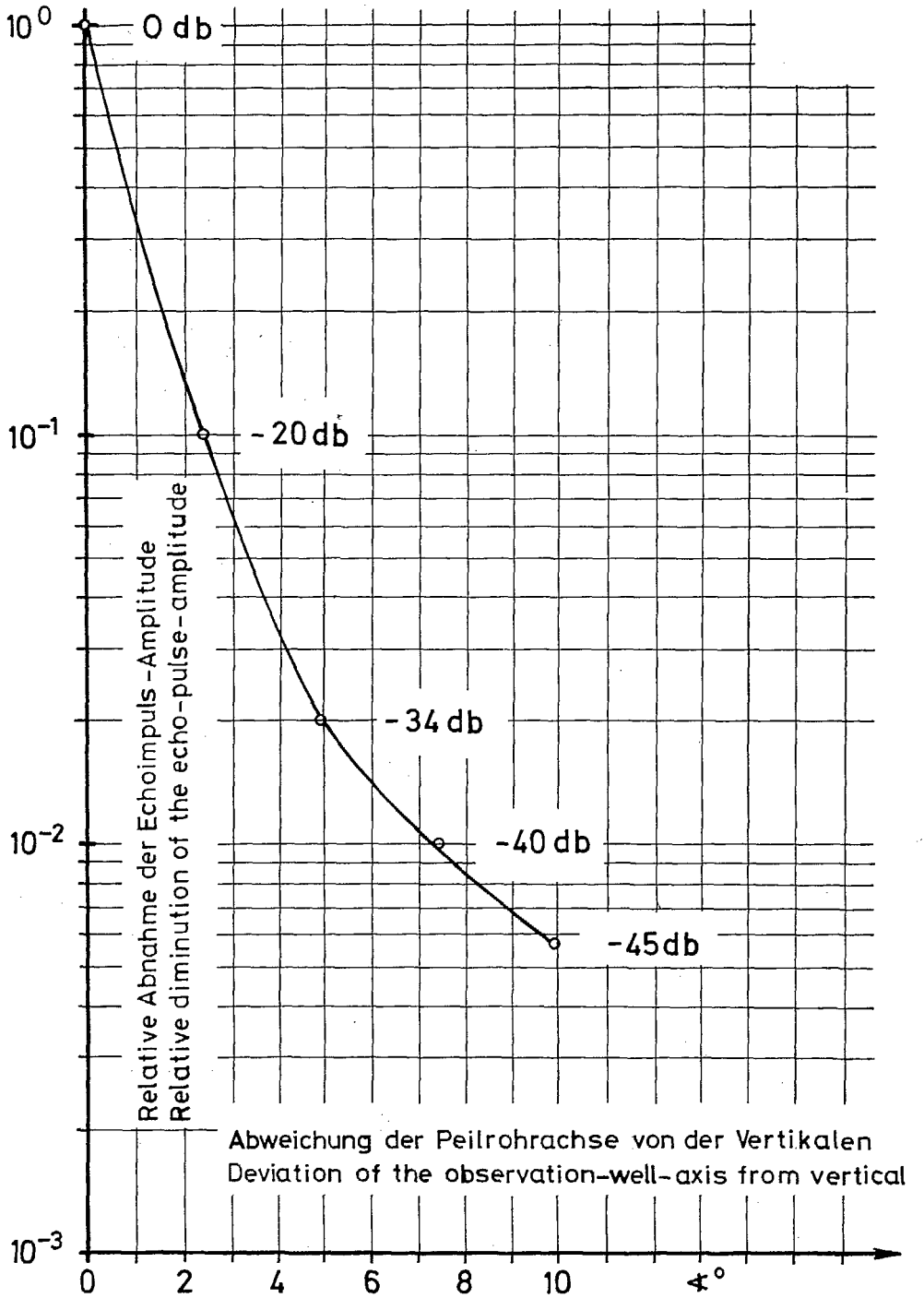


Figure 3. *Deviation of the observation-well axis from the vertical.*

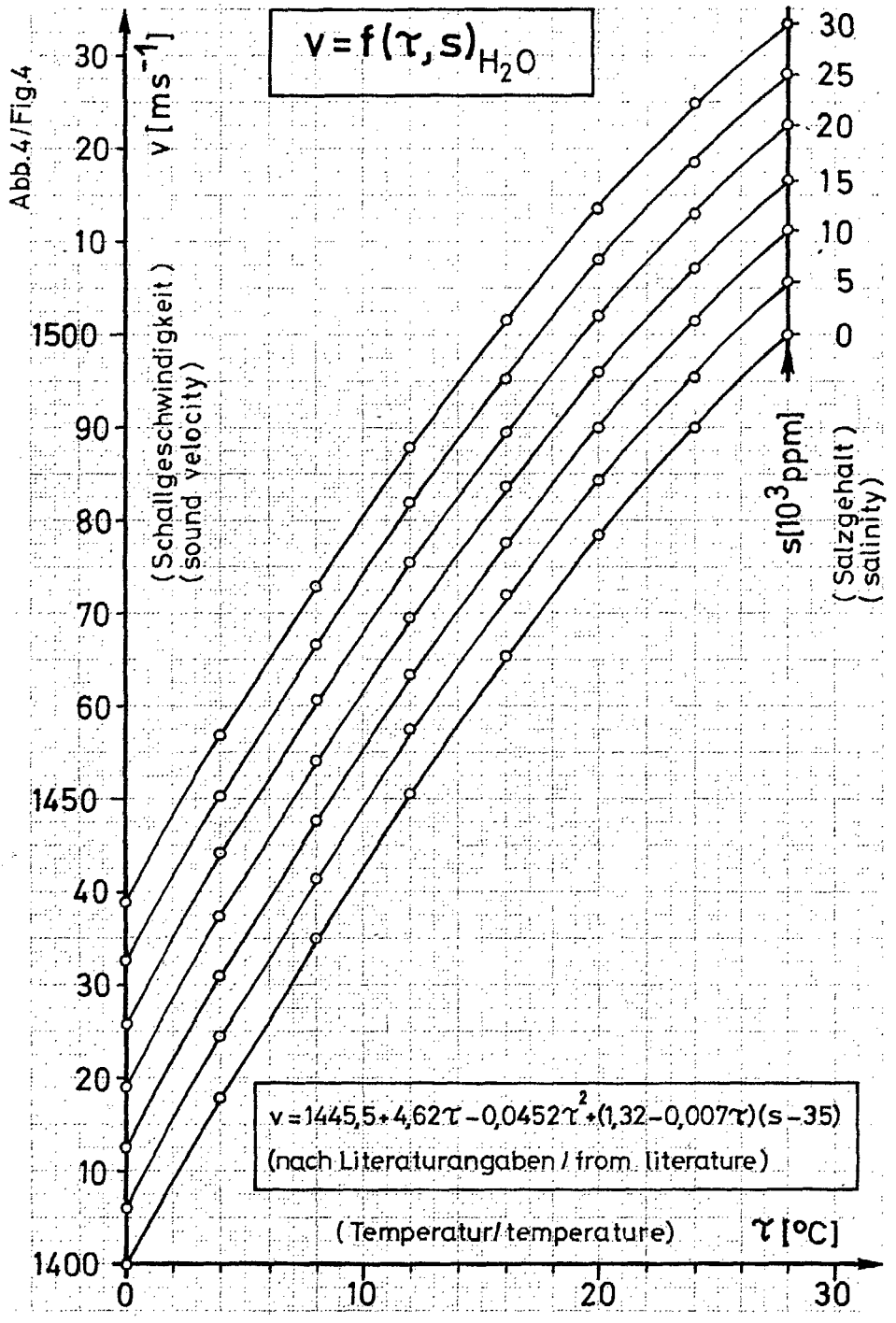


Figure 4. *Temperature/sound velocity.*

4. EXAMPLES

It was stressed in the second section that the acoustic technique makes it possible to obtain a great amount of data at low cost to cover all the interesting phases of a pumping test, to adapt the recording programme to the special conditions and problems inherent in each pumping test, and to store these data in a form which is directly accessible to a computer and to an automatic graph plotter. The interest of such a procedure is demonstrated by the following examples of pumping tests.

Example one stems from an investigation carried out in a sandy, confined or semi-confined aquifer near Meppen, Emsland (Federal Republic of Germany). It is confined by a clayey interglacial layer (Eem), 1—2 m thick, which is present over an extensive area. The vertical permeability of the Eem layer will determine the possibility of recharge from the overlying shallow water table aquifer, consisting of about 20 m fine sands. The drawdown curves $s(\log t)$ (drawdown s recorded at several observation wells, as a function of time t since pumping started, see Fig. 5) show that the water level reaches a quasi-stationary condition after about 100 hours of pumping. Fig. 6 suggests that, as a function of the distance r, the drawdown cone is asymmetrical; this is mainly caused by the drawdown values obtained at well P19. This asymmetric drawdown cone, as well as the quasi-stationary conditions, could have been interpreted at first sight as the non-existence of the confining layer farther to the east. But applying the theory of images to obtain the potential limitations of the aquiclude shows that the data are not consistent with this assumption: the positions of the constructed points are widely scattered, as can be seen from Fig. 6. Plotting the drawdown data as a function of r^2/t on a log-log paper (Fig. 7) and comparing this with the theoretical set of curves for a semiconfined aquifer (Fig. 8) demonstrates that the recharge causing the quasi-stationary conditions should arise from a perceptible vertical permeability of the interglacial material. The deviating behaviour of observation well P19 is therefore probably to be explained by a lower permeability of the aquifer in the surroundings of that well. This is in agreement with the geological findings.

The second example refers to a hydrological investigation in a 60—80 m thick water table aquifer of more or less fine sands in a "geest" area south of Oldenburg in the Federal Republic of Germany. Fig. 9 reveals that, after complex conditions in the first but short phase of the pumping due to local inhomogeneities, we have practically genuine non-stationary conditions with the cone expanding slowly but steadily according to the relatively simple non-stationary differential equations of Theis-Hantush [refs. 1, 2]. Trying to determine the storage coefficient, which in this case should be the same as the effective porosity, from the aquifer behaviour in the first phase of the test does not lead to a reasonable result. Therefore, recourse has been taken to a different procedure including also the data from wells at greater distances and from later times. Differentiation of the Theis-solution:

$$s(r,t) = \frac{Q}{4\pi T} \cdot \int_{\frac{r_s^2 \cdot S}{4Tt}}^{\infty} \frac{e^{-\sigma}}{\sigma}\, d\sigma \qquad (1)$$

where Q = pumping rate, T = transmissivity, S = storage coefficient of the aquifer, $s(r,t)$ = drawdown at distance r from the pumping well at time t after starting the test,

continued on page 28

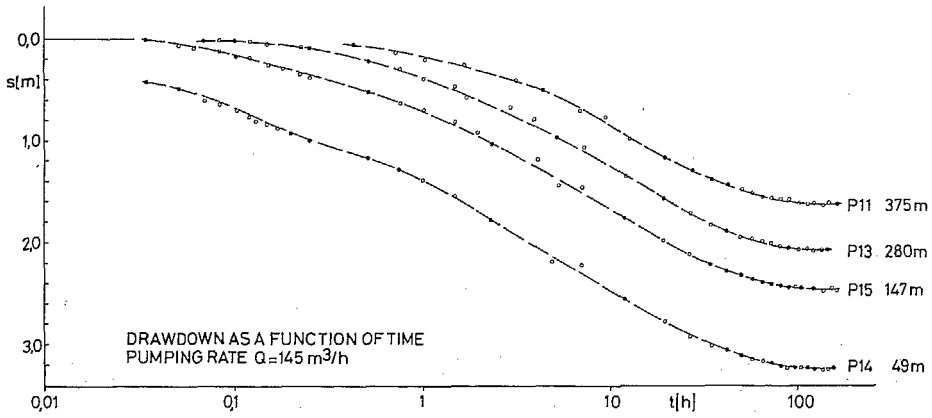


Figure 5. *Pumping test (Schwefingen).*

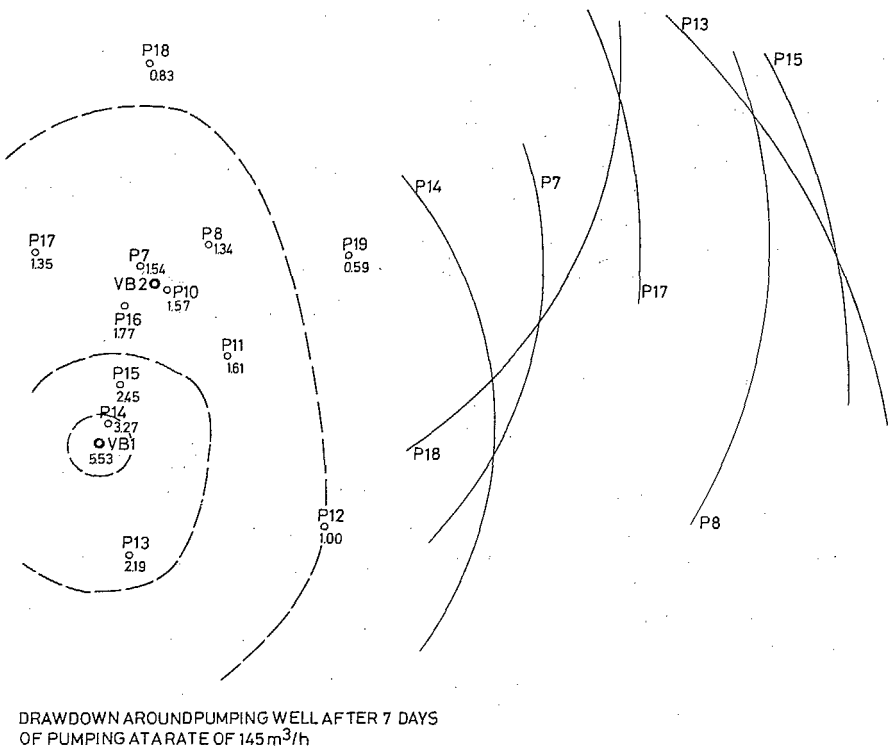


Figure 6. *Pumping test (Schwefingen).*

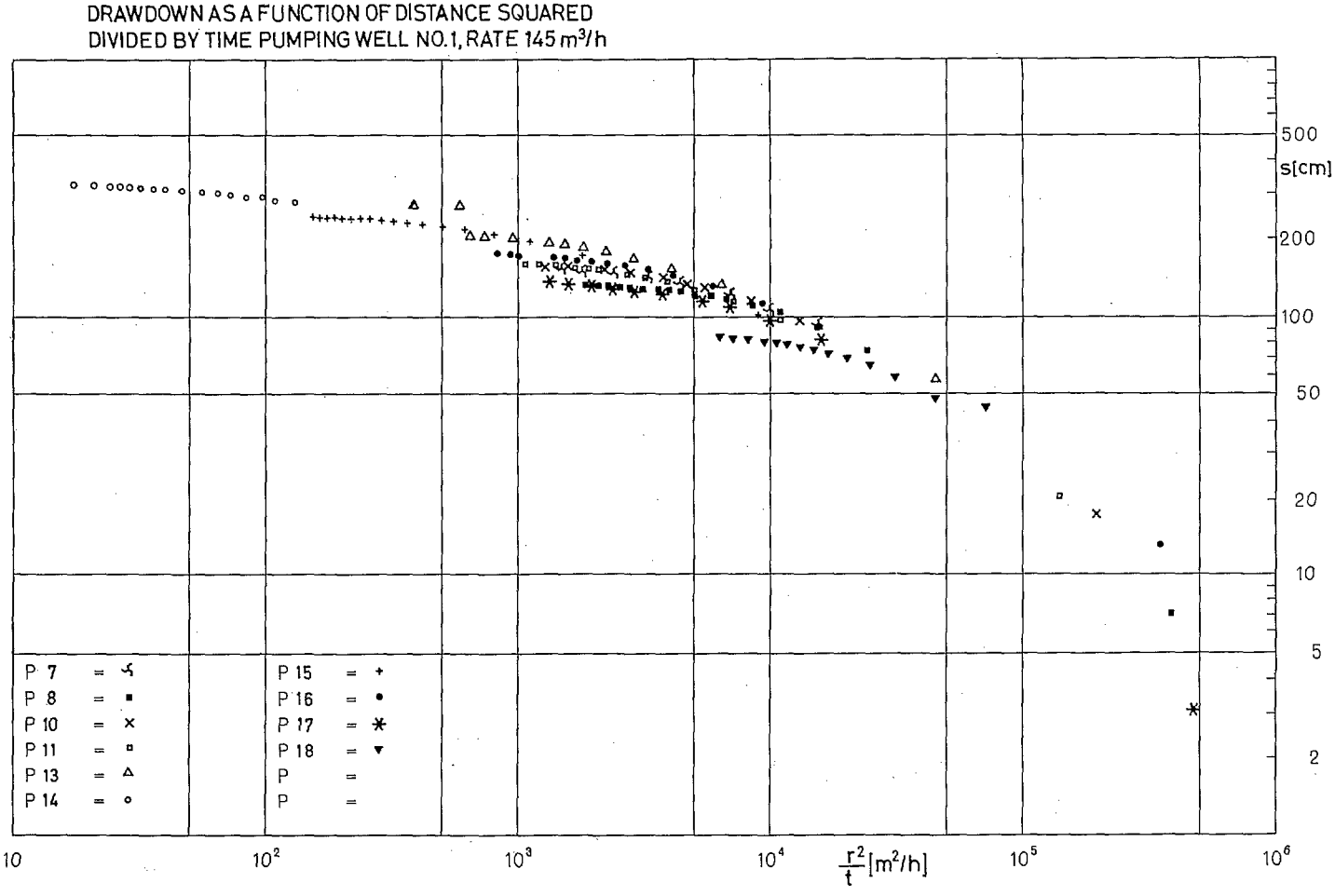


Figure 7. *Pumping test (Schwefingen).*

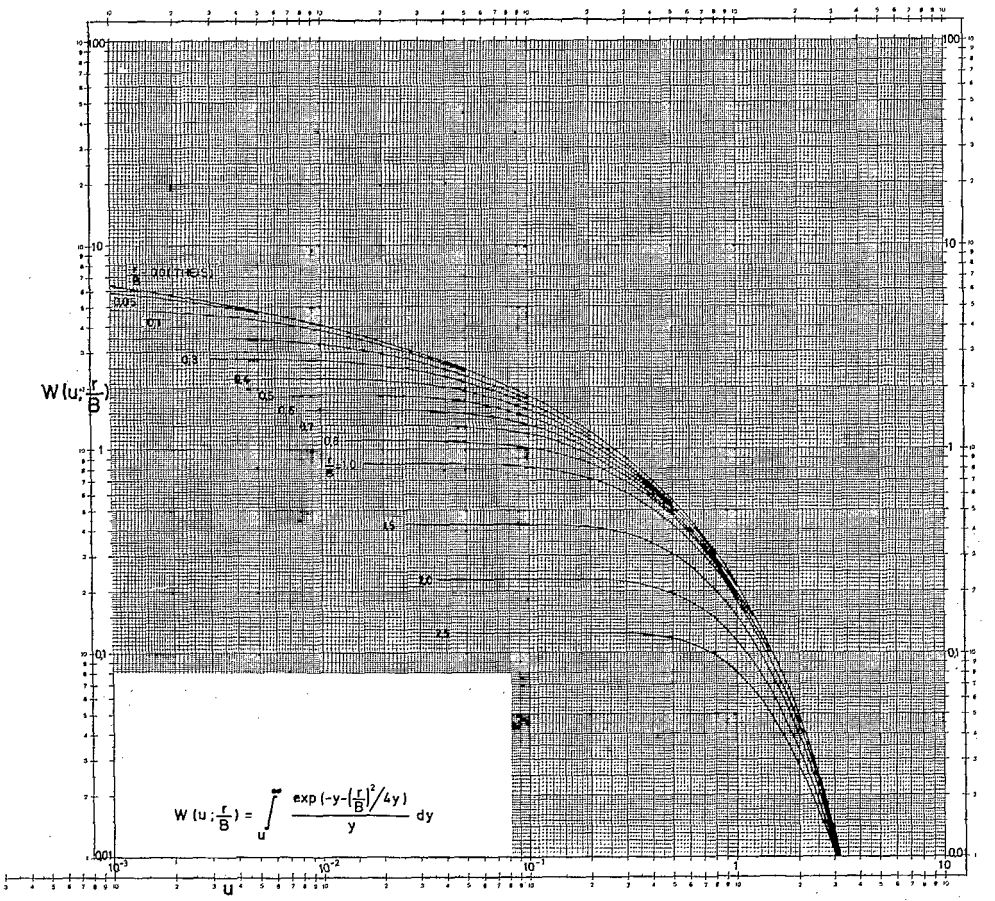


Figure 8. *Well function for aquifer below semipervious aquitard.*

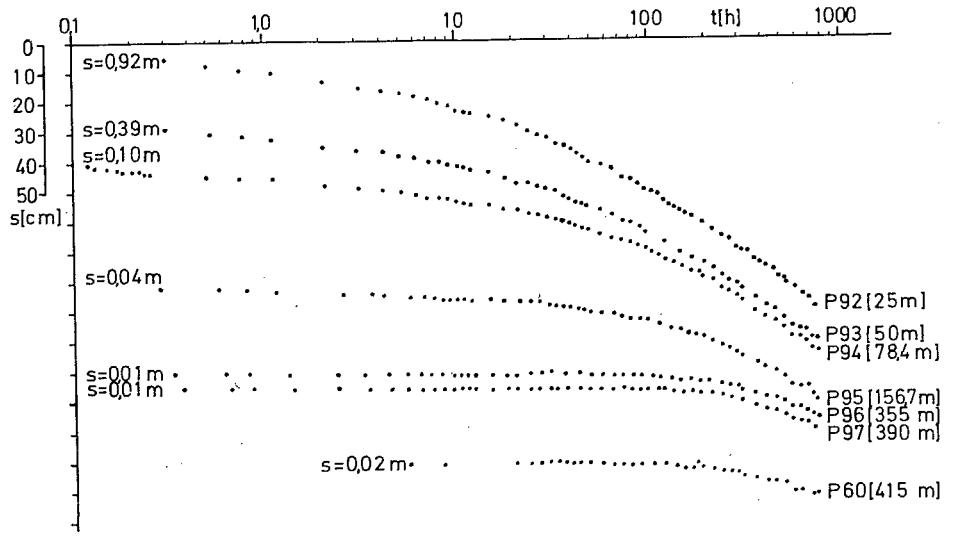


Figure 9. *Pumping test (Greossenkneten).*

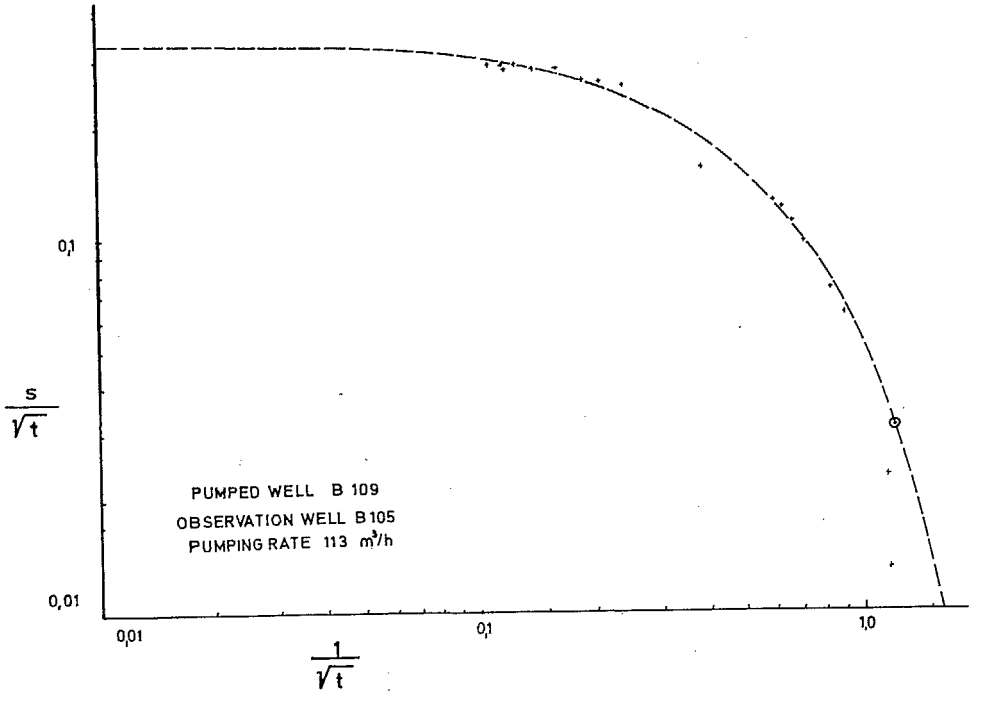
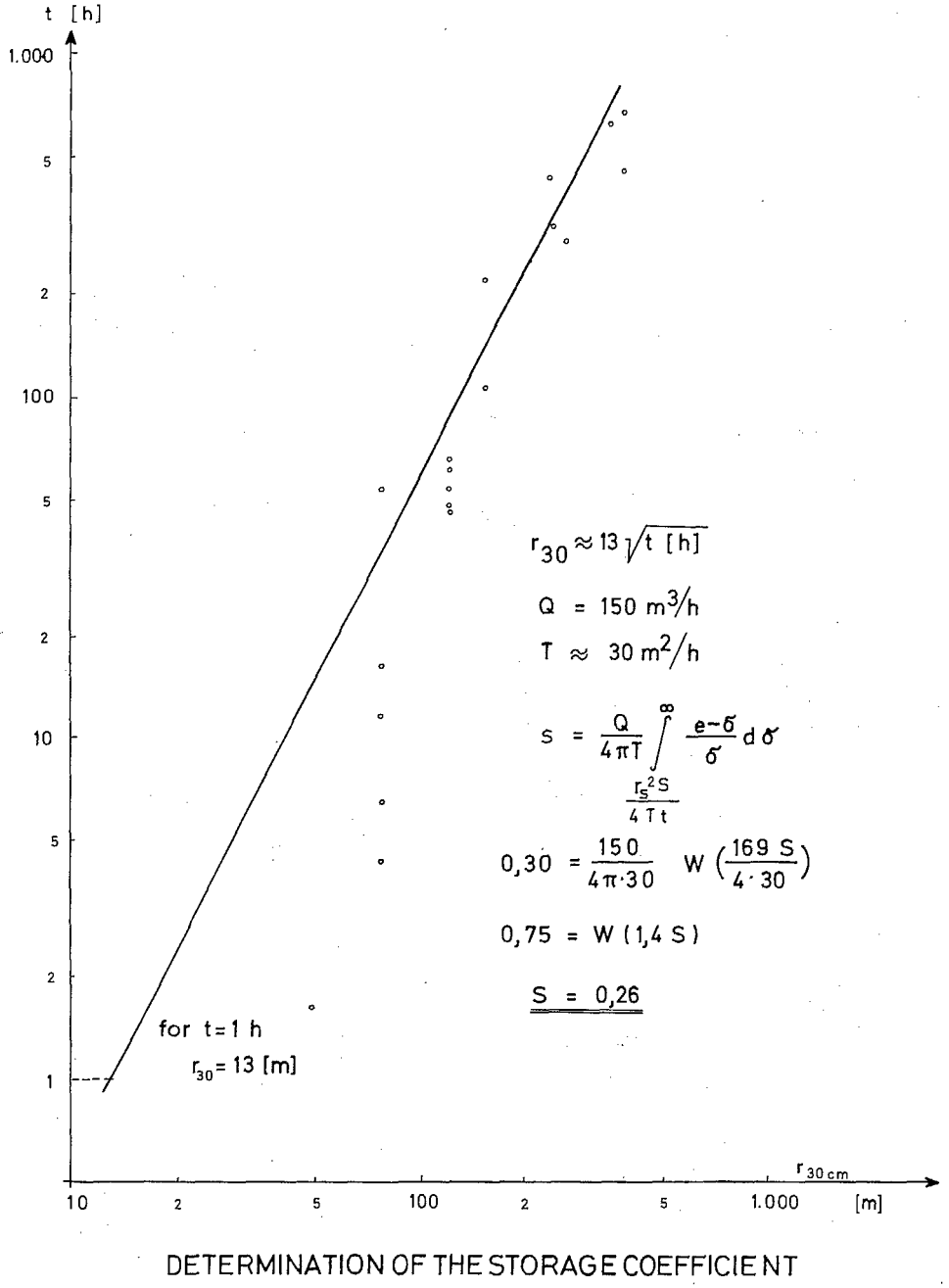


Figure 11. *Pumping test (Arja-Uweina).*



DETERMINATION OF THE STORAGE COEFFICIENT

Figure 10. *Pumping test (Grossenkneten).*

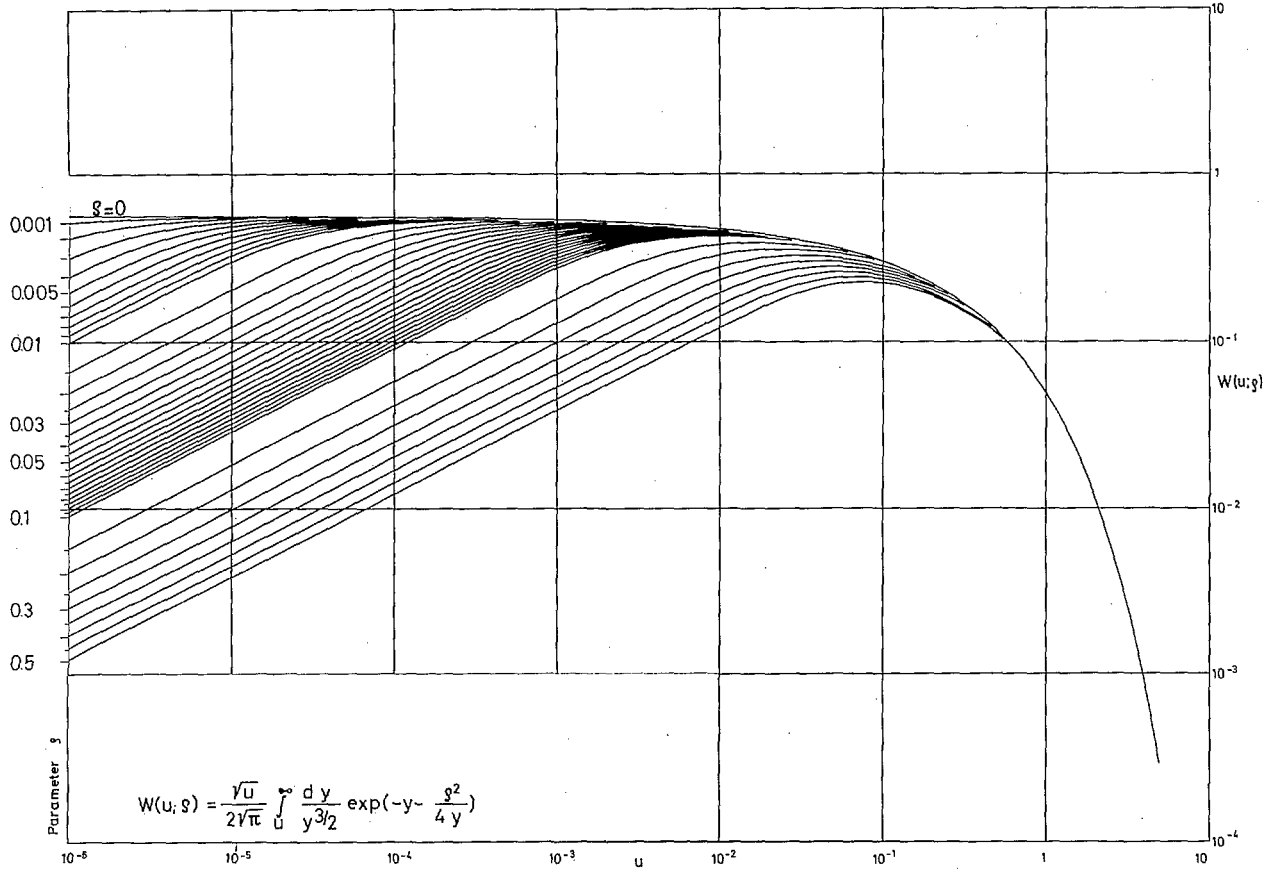


Figure 12. *Well function for joint zone with leakage from low-permeable country rock.*

r_s = distance from the pumping well where the drawdown s is observed at time t with respect to t, leads to the simple differential equation for the distance r_s as a function of time t :

$$\frac{\mathrm{d}r_s}{r_s} = \frac{1}{2}\,\frac{\mathrm{d}t}{t} \tag{2}$$

from which, after integration, the relationship

$$r_s(t) = C_s^* \sqrt{t} \tag{3}$$

is easily derived. Figure 10 demonstrates how the application of equation 3 (transmissivity also derived from the pumping test data) leads to a determination of a value for the over-all "effective" porosity which is of great value in the evaluation of the natural recharge conditions.

The third (and last) example has been taken from a ground-water investigation in the southern Jordan highlands north of Ma'an (Arja-Uweina irrigation project of the Federal Republic of Germany). The aquifer consists of indurated rocks (cherts and limestones) covered by practically impermeable marls. The aquifer outcrops about 7 km west of the well field; thus there are confined conditions. The transmissivity varies widely within the aquifer; the pumping well considered here has been placed in a joint-zone expected to be of much higher permeability than the surrounding parts of the aquifer. There is therefore a close approximation to the conditions of a vertical permeable zone within an aquifer of a low permeability which could be theoretically approximated by a "one-dimensional" aquifer with some leakage through parallel vertical boundaries. With no leakage into the highly permeable joint-zone the problem could be solved by an application of the theory of images. Through continuous reflection of the images at the two parallel boundaries, however, there results an infinite series of images and that solution cannot be utilized for a direct determination of the hydrological constants. In addition, the effect of leakage cannot be taken into account by this theory. On the other hand, the one-dimensional approach leads to a differential equation which can also be solved in the case of leakage into the highly permeable joint-zone. Proceeding in a manner similar to that of Jacob and Hantush in their theory of the leaky aquifer [3], we obtain the differential equation

$$\frac{\partial_s^2}{\partial_x^2} - \frac{s}{B^2} = \frac{S}{T} \cdot \frac{\partial s}{\partial t} \tag{4}$$

$s(x,t)$ drawdown at distance x from the pumping well at time t since starting of the test; S, T as in eq. (1), $B^2 = \frac{K}{k'} \cdot \frac{a \,.\, b}{2 \,.\, \mathrm{M}}$, k' = permeability of the less permeable part of the aquifer; K = permeability, b = width of the highly permermeable zone; M = thickness of the aquifer, a = average distance to the nearly parallel joint-zone. Equation 4 must be solved for the boundary and initial conditions

(a) $$Q = -\,2 \,.\, b \,.\, T \lim_{x \to 0} \frac{\partial s}{\partial x}$$

(b) $$s(\infty, t) = 0$$

(c) $$s(x, 0) = 0.$$

The solution is obtained by the Laplace transformation technique [4]

$$s(x,t) = \frac{Q \cdot x}{4 \cdot \sqrt{\pi} \cdot bT} \cdot \int_{\frac{Sx^2}{4Tt}}^{\infty} \frac{dy}{y^{3/2}} \exp\left(-y - \frac{x^2}{4B^2 y}\right) \tag{5}$$

which for conditions of non-leakage, reduces to

$$\frac{s(x,t)}{\sqrt{t}} = \frac{Q}{b \cdot \sqrt{(S \cdot T)}} \operatorname{ierfc}\left(\frac{x\sqrt{S}}{\sqrt{(4 \cdot T \cdot t)}}\right) \tag{6}$$

where

$$\operatorname{ierfc}(y) = \int_y^{\infty} \operatorname{erfc}(\xi)\, d\xi$$

$$\operatorname{erfc}(\xi) = 1 - \operatorname{erf}(\xi) = 1 - \frac{2}{\sqrt{\pi}} \cdot \int_0^{\xi} e^{-\sigma^2} \cdot d\sigma .$$

For the conditions under which equation 6 is valid, a procedure analogous to Theis's graphical method leads to the determination of a value for the product $b \cdot T$ which could be called the "transmissivity" of the one-dimensional aquifer. One has only to plot

$$\left(\frac{x}{\sqrt{t}} ; \frac{s(x,t)}{\sqrt{t}}\right)$$

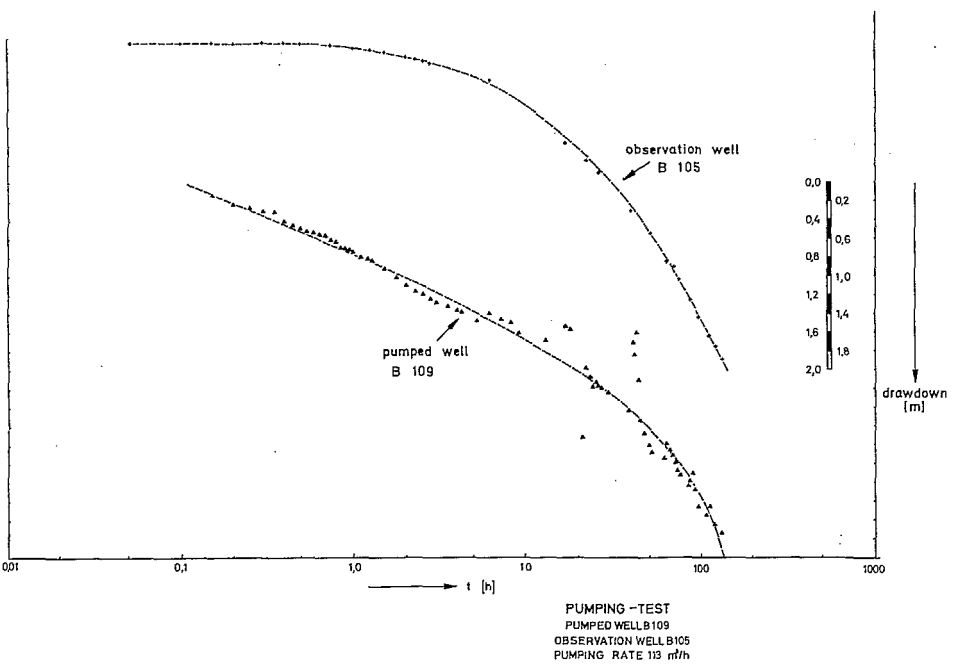


Figure 13. *Pumping test (Arja-Uweina).*

on log-log paper, and to make a least-squares approximation of the "type-curve" $F(u) = \text{ierfc}(u)$, as demonstrated for the data of an observation well at some 500 m distance from the pumping well in our example (Fig. 11). Figure 12 presents the series of standard curves for which the effect of leakage of varying degrees is included. Fig. 13 shows a normal $s(\log t)$ graph of the drawdown at the pumping well and at the observation well considered in Fig. 11. It demonstrates that the data do not fit into the usual scheme of interpretation. The data obtained through these investigations are being used in connexion with a detailed analysis of a long-term pumping test by an analogue model. These three examples should demonstrate the wide range of application of pumping test data in ground-water hydrology.

ACKNOWLEDGEMENT

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BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Theis, C. V. 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using ground water storage. *Trans. Am. Geoph. Union,* vol. 16, p. 519.
2. Jacob C. E. 1963. *Determining the permeability of water-table aquifers.* (Geol. Survey Water-Supply Paper 1536-I).
3. Hantush, M. S. 1964. Hydraulics of wells. *Advances in hydroscience,* vol. 1, p. 281-432.
4. Dürbaum, H. J. 1970. KF-Wert-Bestimmungen aus Pumpversuchen unter nichstationären Bedingungen. In: H. Schneider. *Wasserer-schliessung, 2. Auflage.*

Hydrometric stations in arid zones

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Abstract. The collection of hydrometric data required for national development plans is being pursued vigorously in streams and wadis in Israel. In the arid zones of southern Negev, where many hydrometric stations have been established, serious problems have arisen because of the conditions prevailing in these areas. Visits to these remote measuring sites are possible only in good weather, and even then mostly by air transport. No direct measurements of discharge or manual collection of water samples is possible during flash-floods caused by sudden storms. Sand and dust-storms may influence the performance of the instruments. Bed-load and silt transport clogs the intakes of stilling wells. The equipment and instrumentation should therefore be capable of operating satisfactorily over long periods, unattended and under adverse climatic conditions.

This paper describes equipment and instrumentation developed and used in Israel, such as:

1. A strip chart water-level recorder with battery-driven clock mechanism working unattended for periods up to twelve months. Water-level fluctuations are recorded in a stilling well by means of a float.
2. Same as item (1), but water levels transmitted by electronic means (using stream gauges).
3. Automatic water-sampling bottles filled at different stages on the rising *and* falling limb of the flood-wave.

STATIONS HYDROMETRIQUES EN ZONES ARIDES

Résumé. Les données hydrométriques qui forment la base des projets du développement national sont collectées de façon régulière dans les stations du réseau hydrométrique des rivières et oueds d'Israël.

Dans la zone aride du sud du Néguev où un nombre considérable de stations ont été établies au cours de la dernière décennie, nous nous sommes heurtés à des problèmes sérieux dus aux conditions particulières de la région.

Voici quelques-uns de ces problèmes :

Les stations se trouvent situées pour la plupart en des lieux éloignés, accessibles seulement pendant la saison d'été par transport aérien ; la mesure directe du débit et le prélèvement manuel des échantillons d'eau est impossible pendant les crues soudaines ; les tempêtes de sable empêchent très souvent le fonctionnement régulier des instruments ; les matériaux en suspension ou les transports de fond du lit de la rivière bloquent très souvent les orifices des instruments.

C'est pourquoi il était d'importance primordiale de généraliser l'utilisation des instruments spéciaux qui seraient à même de fonctionner pendant une longue période de façon automatique et qui seraient adaptés aux conditions climatiques difficiles de la région.

Cet article décrit l'équipement et les instruments spéciaux utilisés en Israël :

1. Limnigraphe enregistreur *(strip-chart)* avec un système d'horlogerie fonctionnant sur batterie, automatiquement et pendant 12 mois. Les fluctuations du niveau d'eau sont enregistrées par un flotteur.
2. Appareil identique, mais avec niveaux d'eau transmis par impulsions électroniques *(stream gauges)*.
3. Instrument automatique pour le prélèvement d'échantillons d'eau chargée de matières en suspension – des bouteilles qui se remplissent à des niveaux différents pendant la montée et la descente de la crue.

ESTACIONES HIDROMÉTRICAS EN ZONAS ÁRIDAS

Resumen. En Israel se prosigue intensivamente la recogida de los datos hidrométricos necesarios para los planes nacionales de desarrollo, en todos los ríos y torrentes. En las zonas áridas del Negew meridional, en las que se han establecido numerosas estaciones hidrométricas, se han suscitado graves problemas debido a las condiciones existentes en dichas áreas. Las visitas a estaciones muy alejadas son posibles sólo en el buen tiempo y, en la mayoría de los casos, utilizando transporte aéreo. No resulta posible la realización de mediciones directas del caudal o la toma manual de muestras de agua durante las rápidas crecidas originadas por tormentas súbitas, y las tormentas de arena y polvo pueden perturbar el buen funcionamiento de los instrumentos. El arrastre de fondo y el transporte de sedimentos obturan las entradas de los pozos de remanso. Por tanto era necesario que el equipo y los instrumentos funcionasen satisfactoriamente largos periodos, sin atención y en condiciones climáticas adversas.

El presente trabajo describe el equipo y los instrumentos utilizados en Israel, tales como:

1. Registradores de nivel de agua sobre gráfico de cinta, con mecanismo de relojería accionado por batería que funciona sin ser atendido en periodos de hasta 10 meses. Las fluctuaciones de agua se registran en un pozo de remanso por medio de un flotador.
2. Idéntico al 1), pero con la diferencia de que los niveles de agua se transmiten por medios electrónicos (utilizando medidores de corriente).
3. Botellas tomadoras de muestras de agua que se llenan a diferentes niveles, en el máximo y mínimo de la onda de crecida.

ГИДРОМЕТРИЧЕСКИЕ СТАНЦИИ В ЗАСУШЛИВЫХ ЗОНАХ

Резюме. Сбор гидрометрических данных, требуемых для планов национального развития, энергично продолжается на реках и сухих руслах Израиля. В засушливых зонах южного Негева, где было установлено много гидрометрических станций, возникли серьезные проблемы из-за условий, преобладающих в этих районах. Посещение этих отдаленных измерительных створов возможно только при хорошей погоде и даже в этом случае при использовании воздушного транспорта. Невозможно непосредственное измерение расхода или ручной сбор проб воды во время бурных паводков, вызванных внезапными ливнями. Песок и пыльные бури могут влиять на правильную работу приборов. Перенос донных и взвешенных наносов препятствует наполнению успокоительных колодцев. Поэтому оборудование и приборы должны удовлетворительно работать в течение долгого времени без ухода за ними и при неблагоприятных климатических условиях.

В докладе описываются оборудование и приборы, разработанные и используемые в Израиле, такие как:

1. Самописец уровня води с записью на ленту, с часовым механизмом на батареях, работающий без присмотра в течние 12 месяцев. Колебания уровней воды регистрируются с помощью поплавка.
2. То же (что и (1)) — но уровни воды передаются с помощью электронных приспособлений (используя речные измерения).
3. Автоматические батометры-бутылки, устанавливаемые на разных уровнях, наполняющиеся при подъеме и спаде паводочной волны.

A streamflow measuring station in Israel generally consists of the following equipment and structures:

1. A hydraulic control for low and medium stages of the flow. This may be a measuring weir built specially for this purpose or any structure which proves suitable as a control.
2. A cableway from which current meter measurements of discharge are made to allow calibration of the control (to establish the stage-discharge relationship).
3. A water-level recorder shelter, including stilling well and intake pipe.
4. Gauges and reference marks.

The intake pipes of the stilling wells are cleaned from time to time of the silt which collects in them when a flood wave passes the measuring site. Unless the intake system is cleaned, subsequent records of flow may be lost. Technicians flush the system with water pumped from the stream.

Sometimes conditions prevailing at the site do not allow a current meter measurement to be made: large debris such as trees or tree branches make this measurement impossible because of the danger to the equipment; fine debris gathers on the current meter and stops the rotation of the propeller. In such cases, maximum surface velocity is measured by floats.

Situations arise in which no direct measurement of velocities by current meter or float may be made: the peak discharge may pass some hours before or after the technician visits the site; the approach roads to the measuring site may be cut off by heavy floods; in an extreme case the cableway or other structure may be destroyed by the flood. In such cases, the peak discharge is computed by the slope-area method based on measurements of high-water marks left by the flood.

For small streams inaccessible at the time of floods and heavy rainstorms, structures are built which have been precalibrated in the hydraulic laboratory.

Storm water run-off in a river increases the discharge considerably when a flood wave passes the site. The base-flow after the flood is usually larger than before the flood wave. These waves last for a short period only, usually from one day to a few days at a time. In wadis the flow consists only of flood waves without any base-flow and the streambed remains dry for long periods between floods. It is advisable, therefore, to use water-level recorders working unattended for long periods in order to cut down the number of visits to the site when there is no probability of flood occurrence because of dry weather. Those visits to the measuring site may be not only time-consuming but also costly, especially in the desert; the equipment should therefore operate satisfactorily over long periods unattended and under adverse climatic conditions. Sand and dust storms may influence the proper performance of the recorder and large quantities of bed-load and silt in the water may clog the intakes of the water-level recorder shelters.

These conditions are taken into account and influence the instrumentation and equipment considered for use in the arid zones. If the measuring site may be reached at any time by a good road, then there is no difference between a hydrometric station in semi-arid or in arid zones. Direct measurements of discharge are made by current meter or float and only in exceptional cases will the slope-area method be used to evaluate peak discharges.

If the site is in a remote place with no access to it at the time of flood the following should be considered:

1. *The stage-discharge relation* – as no direct measurement of discharge is possible, a precalibrated hydraulic control should be built or the slope-area method should be used.
2. *Continuous records of stage.* A water-level recorder working unattended for long periods in adverse climatic conditions should be used.
3. *Records of peak flow.* Use can be made of high-water marks left by the flood wave on the banks and/or maximum stage gauges.
4. *Data on silt-content.* As sampling cannot be done by an observer on the site, the installation of automatic silt samplers should be considered.

THE STAGE-DISCHARGE RELATIONSHIP

The precalibrated devices consist of a notch or flume over or through which water passes. These structures are calibrated in a laboratory for certain conditions prevailing in the channel. But these conditions may change when large quantities of silt, sand or gravel are deposited upstream or downstream of the weir. Scour of the bed downstream may change the conditions too, and the stage-discharge relationship may change considerably.

The structure of the weir must be strong to withstand uplift, piping, scour and friction forces of gravel and stones passing over it. Such heavy structures may be very expensive, especially in remote places where difficulties in transportation may make the cost of their erection prohibitive. The changes in channel condition may change the stage-discharge relationship to such an extent that the weir calibration is no longer valid. It is advisable to erect these structures on very important sites only, and in the knowledge that flood frequencies and volume justify the expense and effort. This study can be made from flow records consisting of water-level fluctuations recorded over a number of years and from a stage-discharge relationship based on slope-area measurements.

The slope-area method (sometimes called conveyance-slope method) is a modification of the Chezy-Manning formula of discharge in a natural channel. This formula for steady uniform flow is:

$$Q = \frac{1}{n} \cdot A \cdot R^{2/3} \cdot J^{1/2}.$$

For a given water-level the cross-sectional area (A), the hydraulic radius (R), and the roughness coefficient (n) are constant:

$$K = \frac{1}{n} \cdot A \cdot R^{2/3}$$

$$Q = K \cdot J^{1/2}$$

where K is termed the conveyance, and J is the slope.

The roughness coefficient may be established by comparison with channels for which this value was fixed from measured discharge, slope and area.

The Chezy-Manning formula is for steady uniform flow in which the area and hydraulic radius remain constant along the river channel and the water surface remains parallel to the stream bed. Such conditions do not exist in natural streams: their flow is unsteady and non-uniform. The United States Geological Survey has developed formulae to be used for unsteady non-uniform flow. These formulae are used for evaluation of peak discharges from flood marks left along the banks of the channel. The accuracy of these values is much lower than direct measurements of discharge.

Floods passing a wadi in the arid zone leave flood marks which can be distinguished from previous higher marks if the site is visited a short time after the occurrence of the flood. To ensure the reading of correct high-water marks, the hydrological service in Israel has erected maximum stage gauges on the banks along the reach. These maximum gauges consist of a painted wooden rod 1 m long placed for protection inside a 3 inch (76 mm) steel pipe fixed on the banks. The gauges are

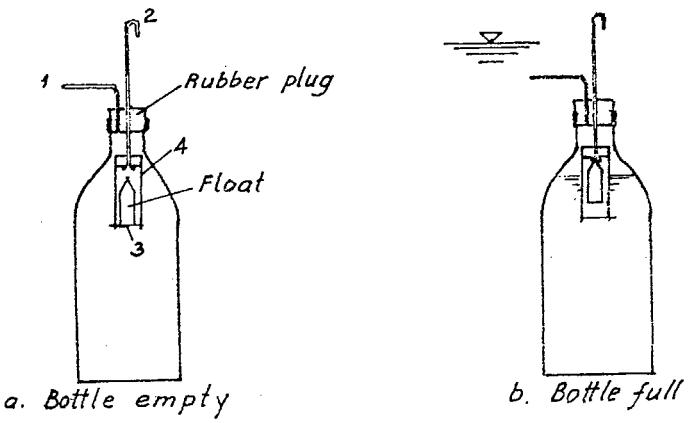


Figure 1. *Automatic silt sampler for the rising limb of the hydrograph.*

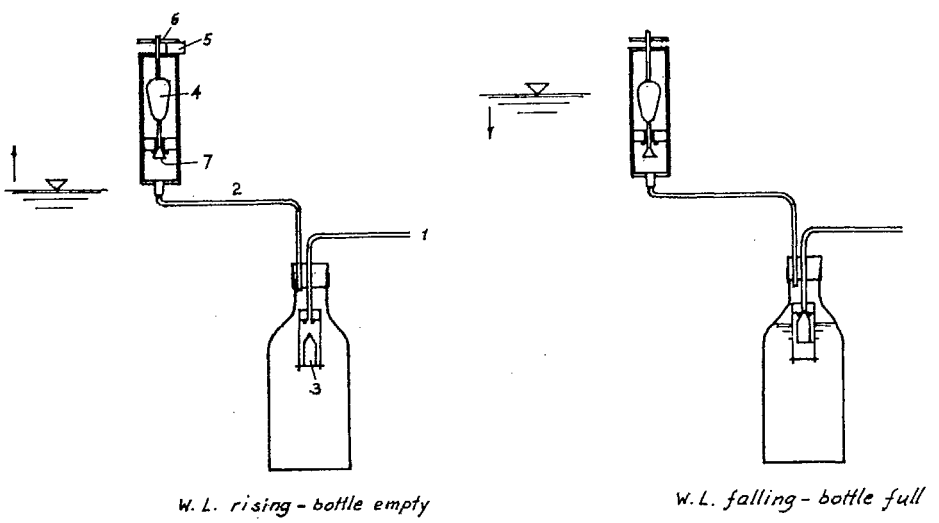


Figure 2. *Improved automatic silt sampler for the falling limb of the hydrograph.*

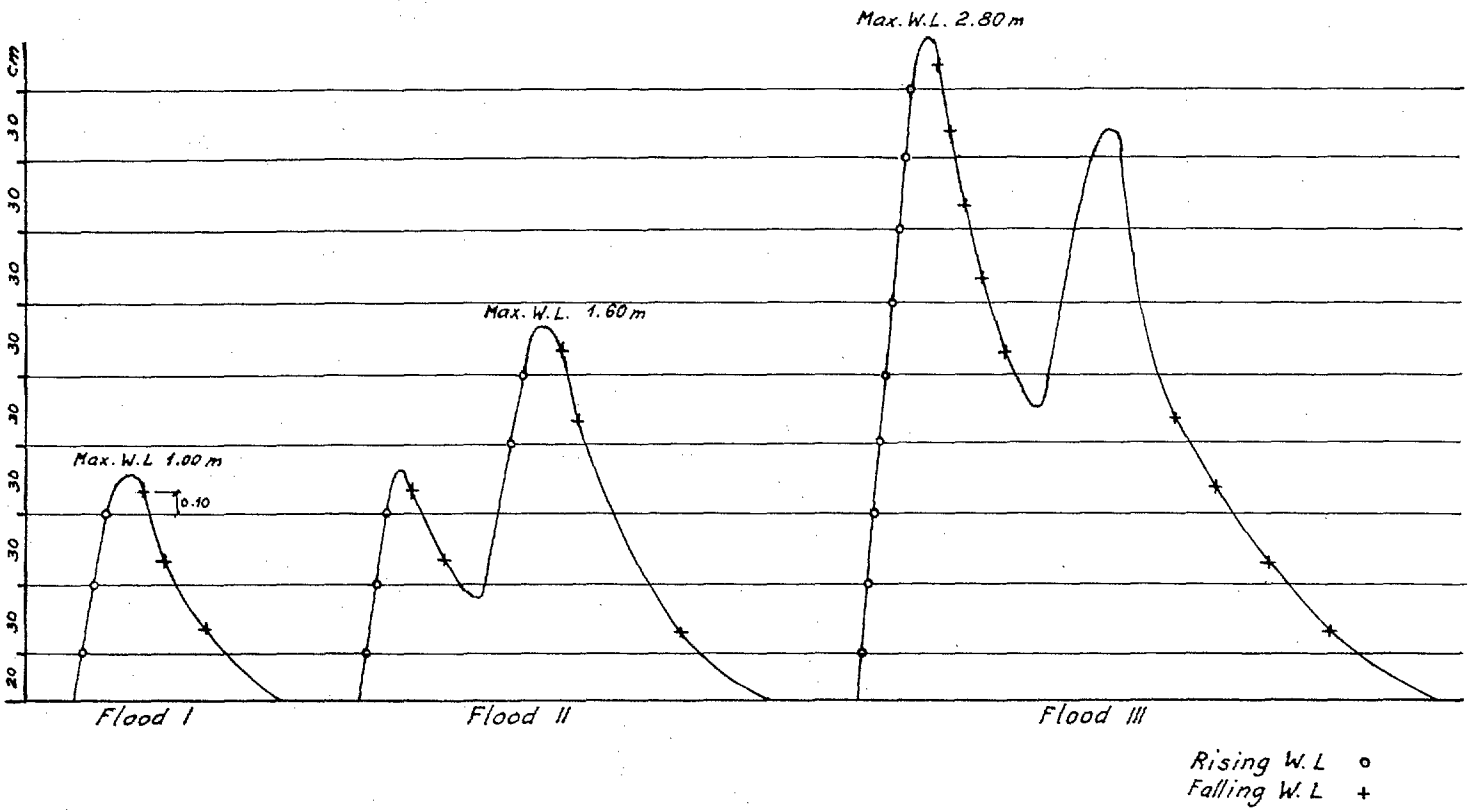


Figure 3. *Automatic silt sampling of three hypothetical flood waves.*

spaced across the bank step-wise (like a stepped staff gauge) to cover the fluctuations from low to highest water level. Water enters the pipe through holes near the bottom and additional holes at the head of the pipe allow the escape of air from the pipe as the water rises. These stepped maximum staff gauges are spaced in cross sections along the reach to allow collection of data for the conveyance slope method.

Continuous recordings of water-level fluctuations are obtained in Israel on strip-chart water-level recorders. The clock mechanism is driven by a battery which provides power for one year's operation. The strip chart is 20 m long, sufficient for one year at a time scale of 48 mm per day. The time scale normally used in the arid zones is 96 mm/day, as flash-floods should be recorded. The strip-chart then lasts for half a year; this is the longest period between visits to the measuring site in a dry year.

The most acute problem in continuous recording is the clogging of intake pipes by large quantities of silt and bed-load moving with the flood water. Unless the intake system is cleaned after each flow event, subsequent records are lost. Flash-floods occur suddenly in the desert, and are often unknown to the hydrological service staff. The hydrological service is working, therefore, on the development of a water-level recorder which will transfer the fluctuations of water level from the river to the recorder by electronic means. A strain gauge installed in the river bed transfers water fluctuations (changes in pressure) to potentiometers which drive the writing mechanism of the recorder. The development of this instrument is well under way. Use of these instruments will make the installation of water-level recorders in the desert much cheaper (no intake pipes and stilling wells being necessary) and will assure continuous records without danger of intake systems becoming clogged by silt.

Silt sampling is usually done by a technician who fills bottles with water from the stream, using auxiliary equipment and instrumentation (silt-samplers, batometers). In the desert (as well as in places remote from settlements) automatic silt-sampling instruments should be installed.

The silt content of the water not only changes with the water level (discharge), but the concentration on the rising limb differs from that on the falling limb for the same water level. The concentration changes with consecutive flood waves. It is therefore necessary to take water samples on both the rising and falling limbs of the flood wave.

Figure 1 shows schematically the instrumentation developed in Israel for automatic silt-sampling on the rising limb of the hydrograph. Water enters the bottle through pipe 1. Air escapes through pipe 2. While the bottle is filling, float 3 rises with the water and closes like a needle-valve thus preventing the escape of air at point 4. This prevents further circulation of water through the bottle and no additional silt will collect in the sampling bottle.

Figure 2 shows an automatic silt sampler for the falling limb of the hydrograph. Water enters through nozzle 1 in a manner similar to that previously described, with the difference that escape of air through pipe 2 is controlled by valve 7. This valve is placed at the lower end of the float 4 and is kept in a closed position by a distance piece, 5, kept in place by plate 6.

The distance piece 5 is made of a substance which dissolves in water (sugar, aluminium sulphate). Hence when the water level rises and reaches piece 5, it dissolves. The float 4 keeps the valve 7 closed and air does not escape, with the result that the bottle remains empty. When the water level falls, float 4 comes down and opens valve 7. At that moment, air escape is allowed and water starts entering the bottle through nozzle 1. Float 3 prevents entrance of water when the bottle is full.

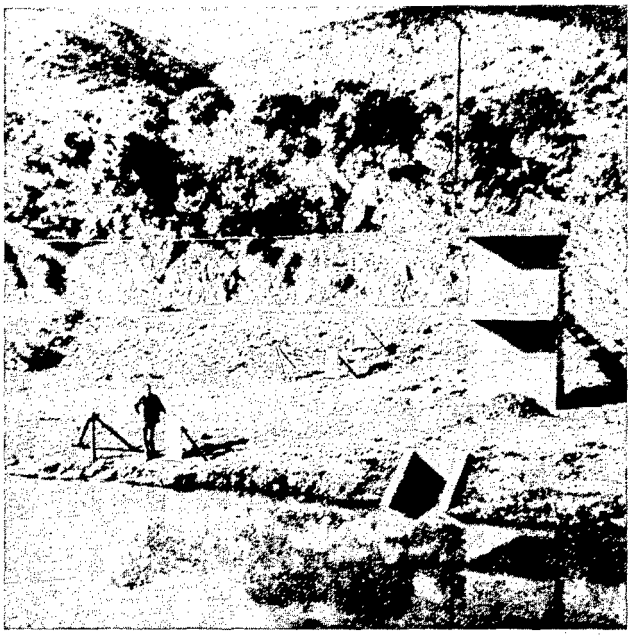


Figure 4. *General arrangements for the silt-sampling shelters.*

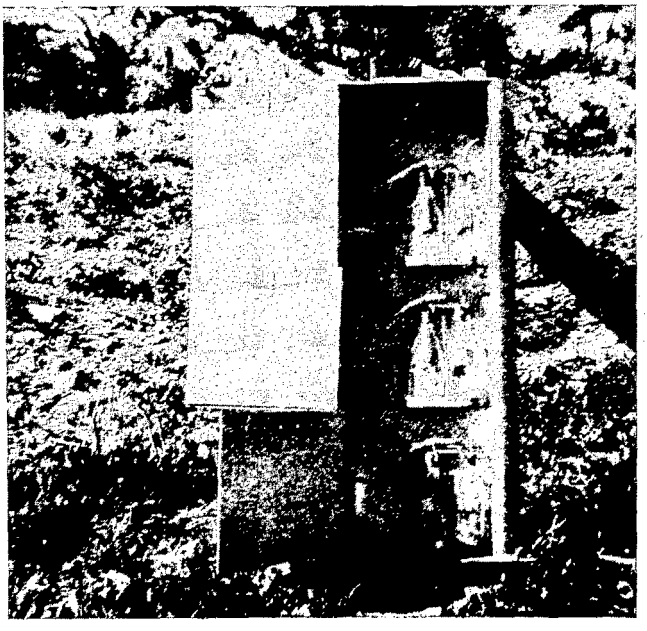


Figure 5. *Interior of a shelter.*

Figure 3 shows automatic silt sampling of three hypothetical flood-waves. Photo 4 shows the arrangement of silt-sampling shelters on the banks of a river in Israel. Photo 5 shows the shelter open.

In conclusion: a hydrometric station in remote places in arid zones contains the following instrumentation and equipment:

1. A water-level recorder working unattended for long periods in adverse climatic conditions. (It is advisable to use pneumatic or electronic instruments which work without intake pipes.)
2. Maximum-stage gauges for collection of data required for the conveyance-slope method of peak discharge measurement and computation.
3. After a few years records have shown that flow magnitudes and flood frequencies make such expensive structures worth while, the erection of expensive precalibrated hydraulic controls should be considered.
4. If silt data are required, automatic silt-samplers should be installed.

The author wishes to thank the Hydrological Service for the material and data put at his disposal in writing this paper.

DISCUSSION

Sherlock: In central Australia we have similar conditions with a number of streams which fall dry from time to time. In a remote area, probably one of the most difficult problems is to measure the water stage. I should like to know about the problems occurring after control weirs have been established. We have met problems in Australia in connexion with the building of control weirs. Concrete control weirs fill up to the top with silt after one or two flows. Further, the whole river bed for many yards back–and upstream–silts up to a level above the weir, affecting its rating or the calibration. We have extreme difficulty in maintaining a stable rating of this type of stream. A flat concrete bed across the bed has been suggested in order not to disturb the river more than necessary. We hope that this will lead to more reliable results. I should appreciate it if Mr. Kornitz would report on the problems in Israel of maintaining a stability of rating or controlling weirs.

André: J'ai eu à travailler en Tunisie et nous avons eu des problèmes à la fois d'ensablement à l'amont et de creusement d'affouillement en aval. Pourrait-on savoir si M. Kornitz a eu le même problème et s'il a essayé de le résoudre en mettant à l'aval des ouvrages en béton, des becs –des sites permettant un réseau de becs– faisant un angle de 22° avec l'horizontal ?

Rodier: Dans les cours d'eau des régions arides (Israël, Afrique du Nord, Australie, etc.) il est très difficile de construire une structure en béton dans le lit de la rivière. Il faut réaliser une station en béton avec des fondations très importantes et avec de très grosses dépenses, sinon l'ouvrage sera emporté dès les premières crues. Il est donc capital de disposer d'une section stable ; mais cela est généralement beaucoup trop coûteux, sauf pour de petits cours d'eau dont les crues ne dépassent pas 50 ou 100 m^3/s, ou lorsque la rivière présente un intérêt économique très grand.

Purich: Would Mr. Kornitz care to comment on the problems of siltation on the upstream reach of the river from the weir due to the construction of the control weirs and the effect of this on the rating of the river?

Kornitz: Because of these difficulties which were also mentioned in the paper, we try at first to collect data for the first estimate by very simple means. This estimate may be only 20 per cent accurate, but we are really glad to have that rate of accuracy which is better than an estimated approach, using the slope method of discharge calculation. A 20 per cent accuracy seems to us quite good, since a weir in the desert with its changing upstream and downstream conditions will never be more than 10 per cent accurate.

In reply to Mr. Rodier's question on maintaining the structures workable at times of floods, I refer to one of the slides of a structure which was destroyed by a flood. This structure was built for 400 m^3/sec maximum discharge. According to an estimate made before it was erected, such a discharge would happen only once in 100 years. But then we had two consecutive years with heavy rainfalls, with a single rainfall of 20 mm in 24 hours on three consecutive days. This caused floods which nobody had expected to happen once in 1,000 years. In fact, we had five big floods which destroyed our structure.

The third question concerned water-level recorders. I mentioned that we are now working on a different kind of instrumentation. These instruments give us the fluctuations by using water levels recorded through a strainer which takes up the water weight or the water pressure on it. We would like to cover them with gravel and stones in the bed, approximately 1 m below the bed level, so that no producing part would be visible in the stream itself. A slide was shown of this instrument, which is only working at present for experimental purposes. It should be borne in mind that in those streams the bottom changes throughout the flood, before the flood and after it. There are pronounced changes in the level and in the configuration of the cross-section. These changes are the reasons for the development of the new device. Such floods in the desert, which may not occur in a dry year or which may occur 5 to 10 times a year for a few days only, call for the development of instruments with a different approach. The constructor of the instruments has further to consider the adverse climatic conditions: sandstorms, silt, and the wide variation of temperature during one day which may amount to as much as 35 degrees.

A portable water-stage recorder for experimental hydrological measurements

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Abstract. Hydrological studies must often be carried out at sites where accurate recording of water-level fluctuations cannot be ensured by the permanent network of water-gauging stations. A light, portable water-stage recorder was used to synchronize experimental measurements of hydraulic parameters on representative river reaches for research on flow conditions in artificial reservoirs and for investigations of glacier run-off in remote areas.

LIMNIGRAPHE PORTATIF POUR MESURES HYDROLOGIQUES EXPERIMENTALES

Résumé. Il est souvent nécessaire de faire des études hydrologiques dans les endroits où les fluctuations de la surface d'eau ne peuvent pas être enregistrées par le réseau permanent des stations limnigraphiques. Un limnigraphe portatif léger a été utilisé pour synchroniser les mesures expérimentales des paramètres hydrauliques dans les sections représentatives des rivières, pour étudier les conditions de l'écoulement dans les réservoirs artificiels et pour examiner le débit glaciaire dans les régions peu accessibles.

REGISTRADOR PORTÁTIL DE NIVELES PARA HIDROLOGÍA EXPERIMENTAL

Resumen. Con frequencia es necesario realizar estudios hidrológicos en lugares en los que las fluctuaciones de nivel de agua no se pueden registrar por la red permanente de estaciones limnigráficas. Se ha utilizado un registrador portátil de nivel de agua para sincronizar las medidas experimentales de los parámetros hidráulicos en secciones representativas, a fin de estudiar las condiciones de flujo en los embalses y examinar la escorrentía glaciar en áreas remotas.

ПОРТАТИВНЫЙ РЕГИСТРАТОР УРОВНЯ ВОДЫ ДЛЯ ЭКСПЕРИМЕНТАЛЬНИХ ГИДРОЛОГИЧЕСКИХ ИЗМЕРЕНИЙ

Резюме. Гидрологические исследования часто проводятся там, где точная регистрация колебаний уровня воды не может быть обеспечена постоянной сеью измерительных станций. Легкий портативный регистратор уровня воды использовался для синхронизации экспериментальных измерений гидравлических параметров на репрезентативных речных участках, для исследования условий стока в искусственных водохранилищах и для исследования ледникового стока в отдаленных районах.

INTRODUCTION

The water stage and its fluctuations must be taken into account in every hydrological measurement on rivers. Routine hydrometry is carried out at permanent water-gauging stations with water-stage recorders. In addition, and especially where the hydrometric cross-section must be moved to some distance for practical reasons,

readings on an auxiliary staff gauge are taken at regular intervals. This practice has been found inadequate for special hydrological studies. Some time ago, a simple portable light-weight water-stage recorder was developed by Javorsky (Hydraulic Research Institute, Prague) in co-operation with Fric. Some recent examples of its application can now be given.

DESCRIPTION OF THE INSTRUMENT

The general appearance of the recorder is shown in Fig. 1. Fluctuations of the water level are followed by a float, which is attached to the lower end of a rod. The float is placed in a perforated cylinder, which is submerged in water. The stilling effect of the cylinder can be adjusted to the respective flow conditions by changing the size of the openings. A pen is attached to the rod, and the water stage is recorded on a revolving drum actuated by a clock. The speed of rotation can be adjusted either to one revolution in 12 hours or one revolution in 6 hours. The rotating drum is protected by a cylindrical cover. The whole equipment is supported by another rod, which is driven into the river bed when the recorder is temporarily installed. All the components pack into a bag which can be carried by one person.

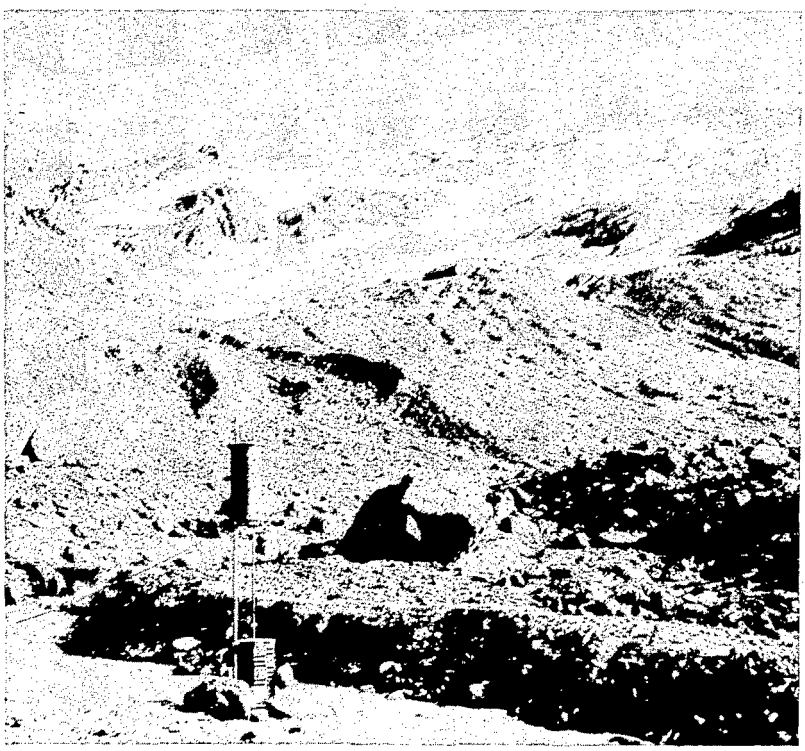


Figure 1. *The portable water-stage recorder near the outflow from the Scaletta Glacier in the Swiss Alps.*

EXAMPLES OF APPLICATION

Studies of run-off in rivers with regard to a possible loss of water due to geological faults [ref. 1] comprised for some time the broadest field of application. In such measurements, a number of portable water-stage recorders are installed at selected intervals along the river, sometimes over a large distance. Water stages are then simultaneously recorded and the discharge evaluated at the respective cross-sections. A decrease of discharge in the downstream direction indicates a loss of water from the river channel.

In studies of flow conditions in backwater reaches, recorders can be installed temporarily along an artificial reservoir formed by a dam. Fig. 2 shows records of water level at different localities on the Slapy reservoir in Czechoslovakia, where periodical standing waves caused by the operation of turbines could also be studied [2].

In studies of velocity losses in rivers due to roughness and channel irregularities [3], it is necessary to measure discharge, sediments, slope and a number of cross-sections, which may take more than one day. Discharge must be expected to change in the meantime, making comparison of the measured values difficult. By using the portable water-stage recorder, it is only necessary to note the exact time of each measurement in order to relate all data, e.g., the cross-sectional areas or water levels of the longitudinal profile, to a reference level for which the discharge is evaluated. An example of this procedure is shown in Fig. 3. If such measurements are

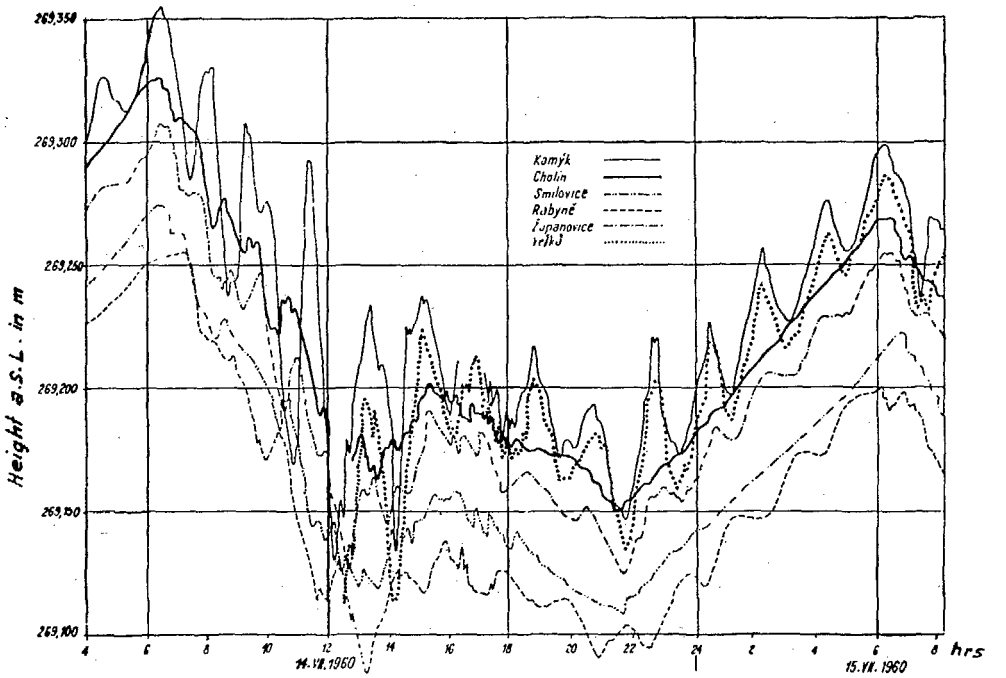


Figure 2. *Fluctuations of water level at different localities along the Slapy reservoir in Czechoslovakia.*

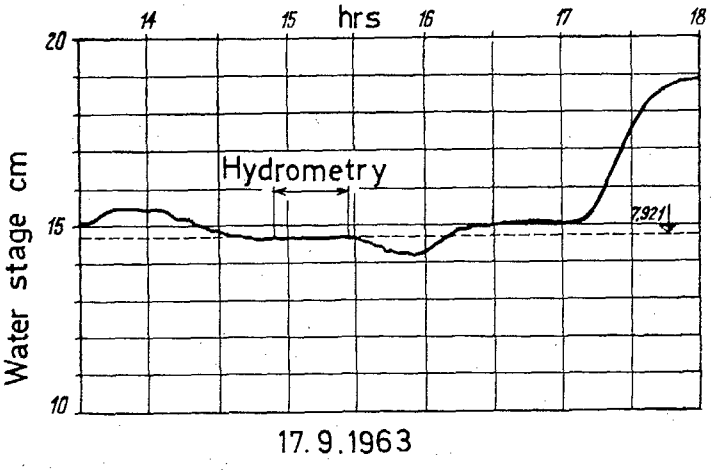


Figure 3. *Water-level recording during the experimental measurement of hydraulic parameters on the river Jizera.*

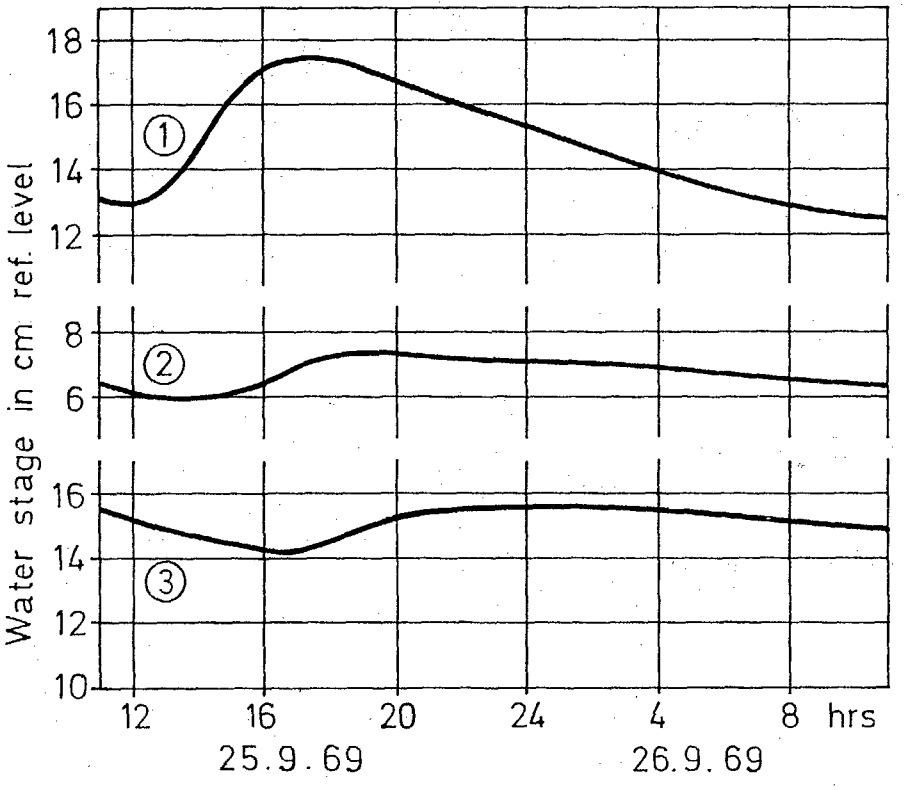


Figure 4. *Comparison of hydrographs of a glacier-fed stream.*

repeated at various water stages, the reference level can be related to a fixed elevation point by levelling.

In some cases, as for example in glacier run-off studies, measurements are carried out in remote areas, where a temporary installation of a regular water-stage recorder would be too expensive and time-consuming. This situation is illustrated by Fig. 1. A comparison of hydrographs along a glacier-fed stream is shown in Fig. 4. The first hydrograph in the vicinity of the glacier was recorded by the portable water-stage recorder, which was installed for 24 hours at an altitude of 2,380 m. Further downstream heavier equipment was installed for several months (hydrograph No. 2). The last hydrograph (No. 3) was recorded by a regular water-stage recorder of the permanent hydrological network.

CONCLUSION

The fluctuating character of water level in small streams, rivers, natural and artificial lakes has to be taken into account, especially in experimental measurements which necessitate high accuracy over a long period. Several examples have illustrated that a light portable water-stage recorder can be of assistance in such cases.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Slepička, F. 1956. Vodnost toku a hydrogeologická stavba území [Runoff in rivers and hydrogeological structure of a region]. *Proceedings 3. Hydrometeorological Conference of 1954, Prague, p. 337-344.*
2. Martinec, J.; Urban, J. 1961. Flow conditions above a high dam. *Proceedings IAHR XIth Convention, Dubrovnik 1961.*
3. Martinec, J. 1966. Rychlostní ztráty v nepravidelných říčních tratích [Velocity losses in irregular channels]. (In Czech with English and Russian abstract.) *Práce a studie VÚV Praha,* vol. 118.

Water-level transducers

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Abstract

I. "Sherlock" unit. An account is given of the design and performance of two transducers that have been developed to meet the rather unusual conditions that are frequently encountered in Australian rivers.

In the Sherlock Pressure Sensing Unit, pressure changes in the system impose a load on one side of a balanced beam. This load is balanced by the movement of a weight on a lead screw actuated by a servo-motor in a follow-up system, which also actuates the recorder.

In its standard form, the unit can handle a range of heads up to 70 feet (21.5 m) with a sensitivity of 0.01 (3 mm) feet throughout the range. Higher range instruments are also available with a corresponding reduction in sensitivity.

II. Irrigation and Water Supply Commission unit. In Australia the variety of the climate, range of river level, the variability of stream flow and the remote locations of many gauging stations present conditions which until very recently were beyond the capacity of any available water-level recording equipment.

In 1960, the Irrigation and Water Supply Commission, Queensland, evolved a gas-electric transducer which has been produced in the Commission's workshops and which with minor modifications, has successfully produced continuous records of river level, over ranges in height up to 100 feet (30 m) in a wide variety of climate conditions, with normal service visits at six-monthly intervals.

The paper discusses the development, manufacture, installation, operation and maintenance of the equipment and also the data produced.

TRANSMETTEURS DE NIVEAUX D'EAU

Résumé

1. Appareil "Sherlock"

Un compte rendu a été établi concernant les détails de construction et le fonctionnement de deux transmetteurs qui ont été conçus pour faire face aux conditions plutôt insolites qui sont fréquemment rencontrées dans les cours d'eau australiens.

Dans le capteur à pression "Sherlock", les pressions transmettent une charge sur l'un des côtés d'une balance. Cette charge est compensée par le mouvement d'un poids monté sur une vis de guidage qui est actionnée par le servo-moteur d'un mécanisme annexe, lequel commande aussi l'enregistreur.

Dans sa forme courante, l'appareil peut être utilisé pour une gamme de charges d'eau allant jusqu'à 70 pieds (21,5 m) avec une sensibilité de 0,01 pied (3 mm) d'un bout à l'autre de la gamme.

Des appareils d'une gamme supérieure sont également disponibles avec une sensibilité correspondante réduite.

2. Appareil de la Commission pour l'irrigation et les ressources en eau

En Australie la variété du climat, la gamme des variations des niveaux des cours d'eau, la fluctuation des débits et l'éloignement de beaucoup de stations de jaugeage créent des conditions qui, jusqu'à ce jour, n'ont pas permis l'utilisation d'un enregistreur ordinaire.

En 1960, la Commission pour l'irrigation et les ressources en eau du Queensland a mis au point un transmetteur gaz-électricité qui a été construit dans les ateliers de la commission et qui, avec de légères modifications, a effectué avec succès et de façon continue des enregistrements de niveaux des cours d'eau pour des gammes de hauteurs d'eau allant jusqu'à 100 pieds (30 m), dans les conditions climatiques très variées, avec des visites d'entretien semestrielles.

Le document décrit le développement, la fabrication, l'installation, le fonctionnement et l'entretien de l'appareil ainsi que les résultats obtenus.

TRANSDUCTORES DE NIVEL DE AGUA

Resumen

I. Unidad "Sherlock". Se describe el diseño y funcionamiento de dos transductores que han sido desarrollados para hacer frente a las condiciones poco normales, que se encuentran frecuentemente en los ríos australianos.

En la unidad detectora de presión "Sherlock", los cambios de presión en el sistema imponen una carga en un lado de una palanca en equilibrio. Este peso se equilibra con el movimiento de un peso sobre un tornillo guía accionado por un servomotor en un sistema secuencial, que acciona también al registrador.

En su forma normal, la unidad puede medir una gama de alturas hasta 70 pies, con una precisión de 0,01 pies en toda la gama. También se dispone de elementos para una gama mayor, aunque con disminución de la precisión.

II. Unidad de la Comisión de Riegos y Abastecimientos de Aguas. En Australia, la variedad de climas, de nivel de los ríos, la variabilidad de caudales en los cursos de agua y los emplazamientos remotos de muchas estaciones medicoras presentan condiciones que hasta época muy reciente sobrepasaban la capacidad de los equipos disponibles para medición de niveles.

En 1.960, la Comisión de Riegos y Abastecimiento de Aguas de Queensland puso a punto un transductor eléctrico de bas fabricado en los talleres de la Comición y que con pequeñas modificaciones ha obtenido con éxito registros continuos de nivel en ríos, para una gama de alturas hasta 100 pies, en una amplia variedad de condiciones de clima, con visitas normales de inspección en intervalos de seis meses.

En esta comunicación se incluye el desarrollo, fabricación, instalación, funcionamiento y mantenimiento del equipo y también los datos obtenidos.

ПРЕОБРАЗОВАТЕЛИ УРОВНЯ ВОДЫ

Резюме I. Прибор « Шерлок ». Дано описание конструкции и работы двух преобразователей, которые были разработаны для необычных условий, часто встречающихся на реках Австралии.

В приборе « Шерлок », чувствительном к давлению, изменение давления в системе изменяет нагрузку на одной стороне уравновешенной балки. Эта нагрузка уравновешивается движением груза по ведущему винту, приводимому в действие серво-мотором в следующей системе, которая также приводит в действие регистратор.

В стандартном виде прибор может измерять уровни в диапазоне до 21 м с чувствительностью в 3 мм на протяжении всего диапазона. Имеются также приборы с более высоким диапазоном, с соответствующим понижением чувствительности.

II. Прибор Комиссии по ирригации и водоснабжению. В Австралии разнообразие климата, диапазон изменения уровней рек, изменчивость стока и отдаленные местоположения многих измерительных станций представляют собой условия, которые до недавнего времени были вне предела возможности любых приборов, регистрирующих уровень воды.

В 1960 г. Комиссия по ирригации и водоснабжению, Квинсленд, разработала газово-электрический преобразователь, который был изготовлен в мастерской комисии и который с незначительными изменениями успешно выполнил непрерывные записи речных уровней в диапазоне изменения их до 30 м, при широком разнообразии климатических условий, с нормальными служебными посещениями один раз в шесть месяцев.

В докладе рассмотрены принципиальная схема, условия изготовления, установки и эксплуатации оборудования, а также получаемая информация.

INTRODUCTION

Rainfall and run-off in Australia are subject to extreme variations in time and place. There are areas of tropical and temperate rain forest and annual precipitation in excess of 150 inches (380 cm) and on the other hand two-thirds of the total area of about 3 million square miles is arid or semi-arid. Rainfalls in excess of 20 inches (50 cm) in one day have been recorded on various occasions in different parts of Australia, and on the other hand large areas may receive no rainfall at all in individual years. Particularly across the north the rainfall is strongly seasonal in pattern, and is concentrated in the summer months.

The variability described above is reflected even more strongly in stream flow characteristics. Ratios of the lowest to the highest annual discharge of the order of 1 to 300 are not uncommon, and seasonal conditions are such that large rivers in the north of the continent which may carry upwards of 1 million cusecs (30,000 cubic metres per second) in high flood, may cease to flow altogether in the dry season. Flood heights of about 70 feet (21 metres) are not uncommon on major streams, and heights of over 100 feet (30.5 metres) have been recorded on at least two occasions. Most of these streams have moveable beds and during floods, velocities in excess of 15 feet per second (4.5 metres per second) are not uncommon. The rate of rise of such rivers may be up to 20 feet per hour (6 metres per hour) and as a result the rivers carry large quantities of suspended sediment and bed load.

These conditions obviously impose great problems in connexion with the construction and operation of float-actuated recorders, and for this reason there were two independent moves in Australia to develop transducers which would enable good-quality data to be obtained in the wide ranging and adverse conditions which must be provided for. Although the two organizations worked to somewhat different performance specifications, the following summary could be taken as applying generally in both cases, with minor exceptions which are referred to in the individual papers.

Stream conditions

Range	0 to 100 feet of water (0 to 30.5 metres)
Rate of change	20 feet (6.1 metres) per hour

Ambient conditions

Temperature range	5°F to 150°F (−15°C to 65°C)
Humidity	Up to 100 per cent

Tropical conditions of corrosion and mould may exist and on the other hand bad dust conditions may occur.

Instrument

Accuracy	0.01 feet (3 mm)
Torque	At least 20 ounce inches (1440 g.cm) sufficient to operate two conventional recording or transmitting instruments
Electrical	To operate for at least six months on a six-volt dry battery
Reliability	To operate without attention for at least three months

Physical features	To be compact, light-weight, easy to install and operate, and as rugged as possible in view of need for installation in remote locations.

The instrument must not only perform as required, but it must be readily shown that it is doing so. This has several aspects including

1. The instrument accuracy should be maintained as the rate of river stage height increases, since measurements are more important and are harder to check under these conditions.
2. It should be possible for field staff to carry out full testing of the instrument on site, at least as regards the performance of its main components.

The installation must also avoid any exposed elements which are likely to be damaged by vandals, or carried away by floods or debris in the river. In meeting all the above requirements, the designer must never lose sight of such basic qualities as simplicity, low cost, and output suitable for real time use as well as for recording and automatic data processing.

I. SHERLOCK PRESSURE SENSING UNIT

by D. J. Sherlock

Concept of unit

The cost of installation of float-operated recorders prevented river-stage level recording of many of the larger Australian streams and it was not until the mercury manometer was adapted for this purpose that it became feasible to instrument these larger streams.

However, a need still existed for an "intermediate" instrument, one that was possibly cheaper and easier to install than the manometer-servo and would operate on either a bulb (closed) or bubble (open) pressure system.

About mid-1965, an investigation was undertaken of existing pressure measuring and allied instruments and it was concluded that an instrument based on a pressure capsule offered the greatest potential. Pressure bellows technology had advanced greatly with the advent of space research and highly stable bellows were becoming available.

Various pressure-bellows configurations were considered and it was decided to adopt the force-balance method because of its greater inherent accuracy and its relative independence of those bellows characteristics which change with either pressure or temperature. Also differential measurements could be accommodated by mounting two bellows in an opposed fashion.

Accordingly a concept of a pressure sensing unit evolved that had provision for measuring differential pressures up to 30 p.s.i. (70 ft. head of water) (2.1 kg/cm^2), hence the name Sherlock Pressure Sensing Unit Model DP 30. However, as most applications required simple measurements, only the single bellows instrument has been developed at this stage.

Force-balance system of pressure measurement

In practice, the force-balance system involves the measurement of the force required to restore a flexible pressure bellows to its normal or "null" position after pressure has been applied. The magnitude of the force is equal to the pressure applied times the effective area of the bellows and provided the bellows area remains constant, pressure is proportional to the restoring force.

The application of this technique to the Sherlock Pressure Sensing Unit is illustrated in Fig. 1(a) and 1(b).

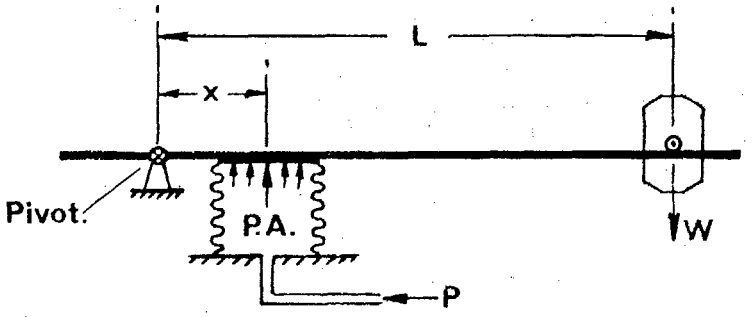


Figure 1(a). *Force output balanced by lever-mounted system so that*

$$P.A.x = W.L$$

where P = *pressure applied*
A = *effective area of bellows*
W = *balancing weight*

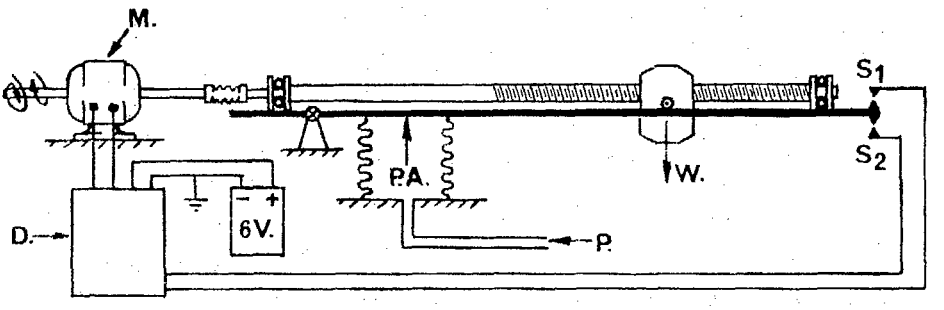


Figure 1(b). *Force output balanced automatically by servo system. An increase in pressure causes beam to rotate and close switch S_1 which in turn operates the motor (M) through the delay unit (D). The motor continues to turn until switch S_1 opens and beam has been restored to the null position. A decrease in the pressure causes switch S_2 to close which moves weight in opposite direction. Hence the angular rotation of the motor shaft is a precise measure of the pressure applied.*

Feasibility study

Before embarking on this study, a schedule of performance and design requirements was prepared. Calculations were then made on an energy input-output basis and on

work-done principles and the general feasibility was verified. Also, these calculations provided basic guidelines for the design and it was now evident that the design must incorporate the following:

1. A bellows of comparatively large area to provide the necessary input energy plus a reasonable allowance for reliability.
2. A balanced beam assembly of (relatively) high inertia to damp out vibrations and spurious pressure fluctuations.
3. The flexing components (the bellows, pivots and coupling) to impart the minimum restraint to the movement of the beam.
4. The dynamic effect of the servo mechanism on the beam to be small in relation to the inertia of the beam.

It now had to be determined whether these requirements would increase the price of the instrument beyond an acceptable level. Whilst there is probably a universal tendency for a designer to underestimate production costs, an analysis indicated that production costs would not be too high and certainly less than a comparable manometer-servo unit.

Design and development

It was decided initially to follow a part design-part prototype approach but it was soon found that it failed to produce worth-while results. It appeared that the interrelation of the components was such that it was necessary to make almost an entire instrument to the tolerances suggested by the calculations to ascertain performance levels. Accordingly the instrument was designed almost in its entirety before a working prototype was constructed.

Obtaining components that would meet the specification posed many problems to the point where, in the case of the bellows, it was necessary to provide the manufacturer with technical assistance.

With the solving of each component problem, the performance of the prototype was improved, until the stage was reached early in 1967 whereby the performance specification, as laid down, had been achieved. Demonstrations followed and in May 1967 the prototype was tested by a local water authority, the Sydney Metropolitan Water, Sewerage and Drainage Board. The test involved installation at a gauging station for a period of six weeks and comparison of results with an adjacent float recorder. Both open and closed pressure systems were used.

The results obtained, which were generally satisfactory, were made available to other Australian water authorities, most of which agreed to purchase an instrument for evaluation. A "pre-production" batch of twenty instruments was undertaken and the first of these delivered early in 1968. In most aspects the performance attained with these pre-production instruments was better than anticipated but many modifications were found necessary. With some instruments, the bellows exhibited instability, making the instrument inaccurate over a portion of its range. The problem resolved into altering the bellows design but a great many trials were required to produce a satisfactory result.

Finally it was considered that sufficient information had been obtained and a design for a production model was adopted. Production commenced about mid-1969 and the current instrument is depicted in the photographs which follow.

Installation and operation

The importance of correct and thorough installation of hydrological instruments cannot be overstressed. Every effort must be made, firstly, by the instrument designer to ensure that the instrument is relatively simple in its operation and secondly by the operating authority to provide adequate "on site" facilities.

A well ventilated walk-in shelter is recommended. The platform supporting the instrument should be rigid, and provision should be made for mounting the bubble unit and the gas bottle. In hot climates, shelters painted white are preferred for maximum reflection of the sun's rays.

The instrument is designed to accept pressure tubing of either 3/16″, 1/4″ or 3/8″ outside diameter. Heavy-walled polythene tubing 3/8″ OD × 1/8″ ID is the most popular as it can be laid directly into the ground. Protection in galvanized iron water pipe is suggested in exposed sections and down steep banks. The tubing is terminated in the stream bed so that it will not move. A typical installation is illustrated in Fig. 4.

Each pressure connexion must be made as gas-tight as possible. Very minor leaks can be tolerated with a gas purge system but gas consumption will be higher. When operating with a bulb, the pressure must be completely leak-free, as there is no replenishment of gas. However, as there are only two connexions involved – the instrument and the bulb – trouble in this regard is not common.

It is considered good practice to treat each joint with a soap solution, the formation and bursting of bubbles indicating a leak. Proprietary leak-detecting solutions are more expensive but are more sensitive to very small leaks. Once a system has been made gas-tight, it usually remains trouble free unless violently disturbed in some way.

Best results are obtained using a bubble unit, but because of its simplicity the bulb is sometimes preferred and the variations with water temperature accepted.

Note: Instruments must be calibrated for bulb operation because of a small loss of effective head within the bulb.

The recommended method of driving the primary recording instrument is by gears as shown in Fig. 5. An additional instrument can be chain driven from a sprocket mounted on the output shaft behind the spur gear.

A large (220 cubic feet) (6,2 m^3) bottle of nitrogen with standard gas regulator is normally used. The bubble rate can be set to the site conditions (i.e., length of tubing and maximum likely rate of rise) but a standard bubble rate of 150 per minute is suggested for the Conoflow regulator.

When the servo motor is operating, there is a current drain of 40 milliamps at 6 volts. A large 6 V dry battery of 10 ampere-hour capacity will operate the instrument for more than twelve months, but annual battery changes are recommended.

The use of pre-lubricated bearings obviates the need for regular oiling, and routine maintenance only involves the removal of dust from any moving parts and the cleaning of the electrical contacts.

The mortor control unit, which amplifies the switching current and provides a choice of delays before the motor is switched on, is a printed circuit transistorized device. Whilst this unit is highly reliable, it has been made as a plug-in unit, so the whole unit can be replaced if necessary. The actual circuit uses popular components which should be available in most countries.

continued on page 57

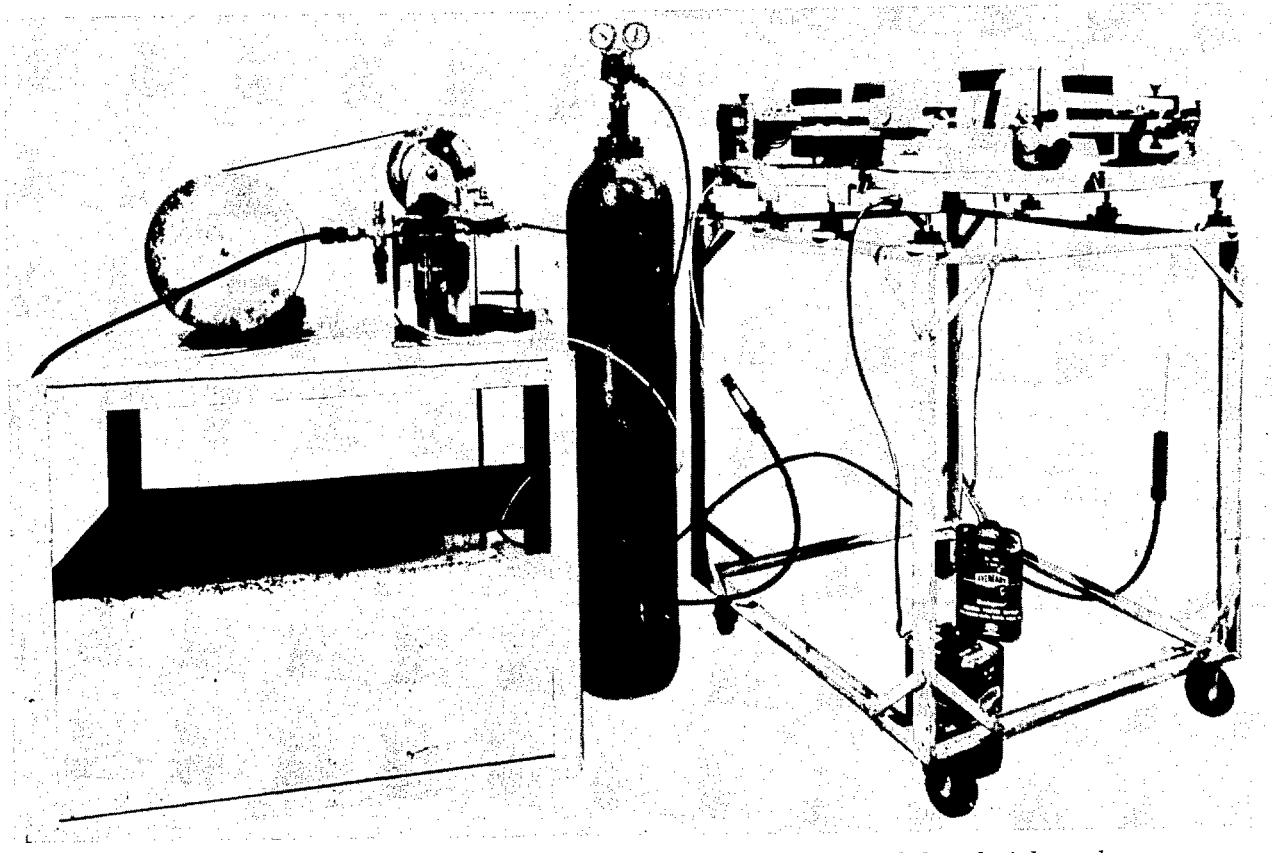


Figure 2. *Two DP 30's being calibrated for gas purge operation. The bubble outlet is lowered down a sealed pipe (70 feet deep) filled with water and the weight of the top and bottom sections of the balancing weight reduced until the instrument agrees with tape gauge attached to the bubble outlet. The popular Conoflow bubble unit is used but without the normal shut-off valves*

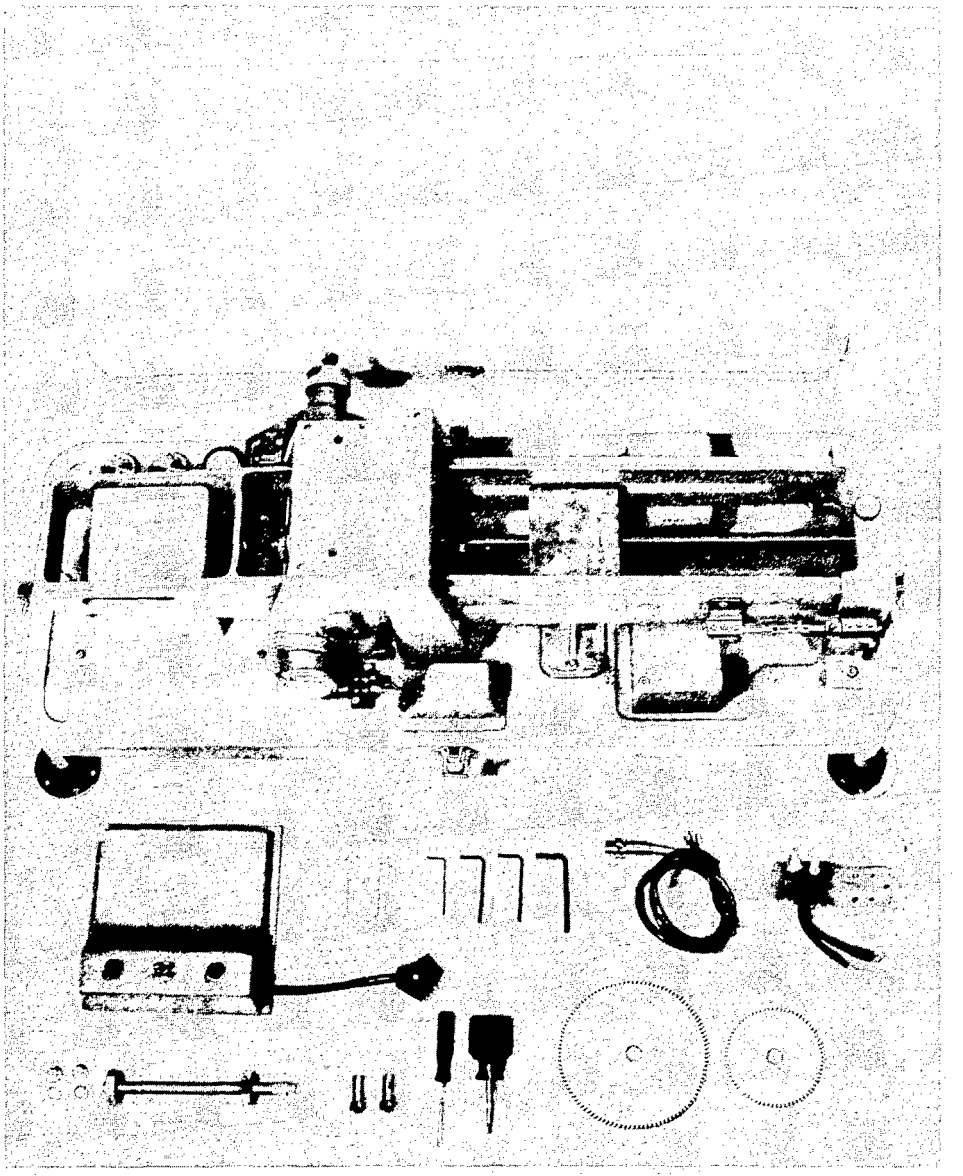


Figure 3. *The DP 30 ready for packing in carton. The weights are removed from beam and screwed to the base, the balancing weight nut and lead screw are secured to a cast-aluminium transit clamp. The Mercoid "back up" switches and the control box shown in foreground are packed separately but the normal relay type contacts remain fitted*

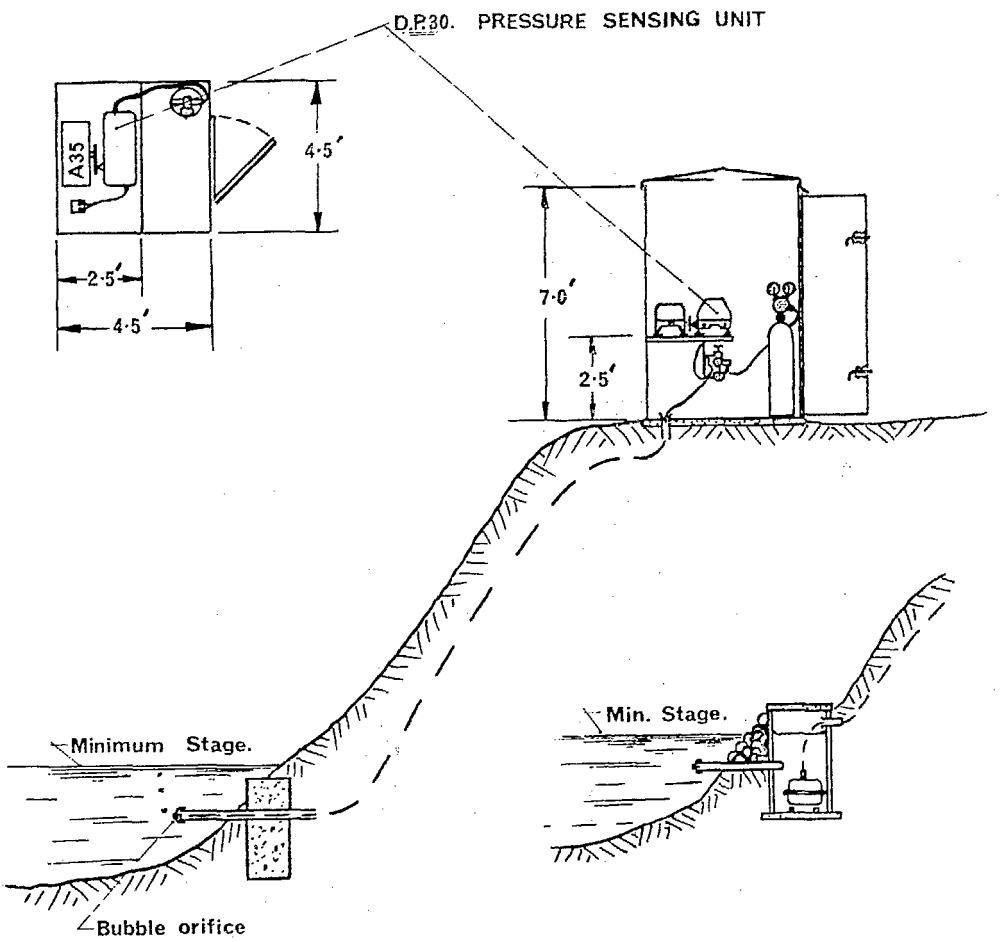


Figure 4. *Typical installation using a bubble unit or a pressure bulb*

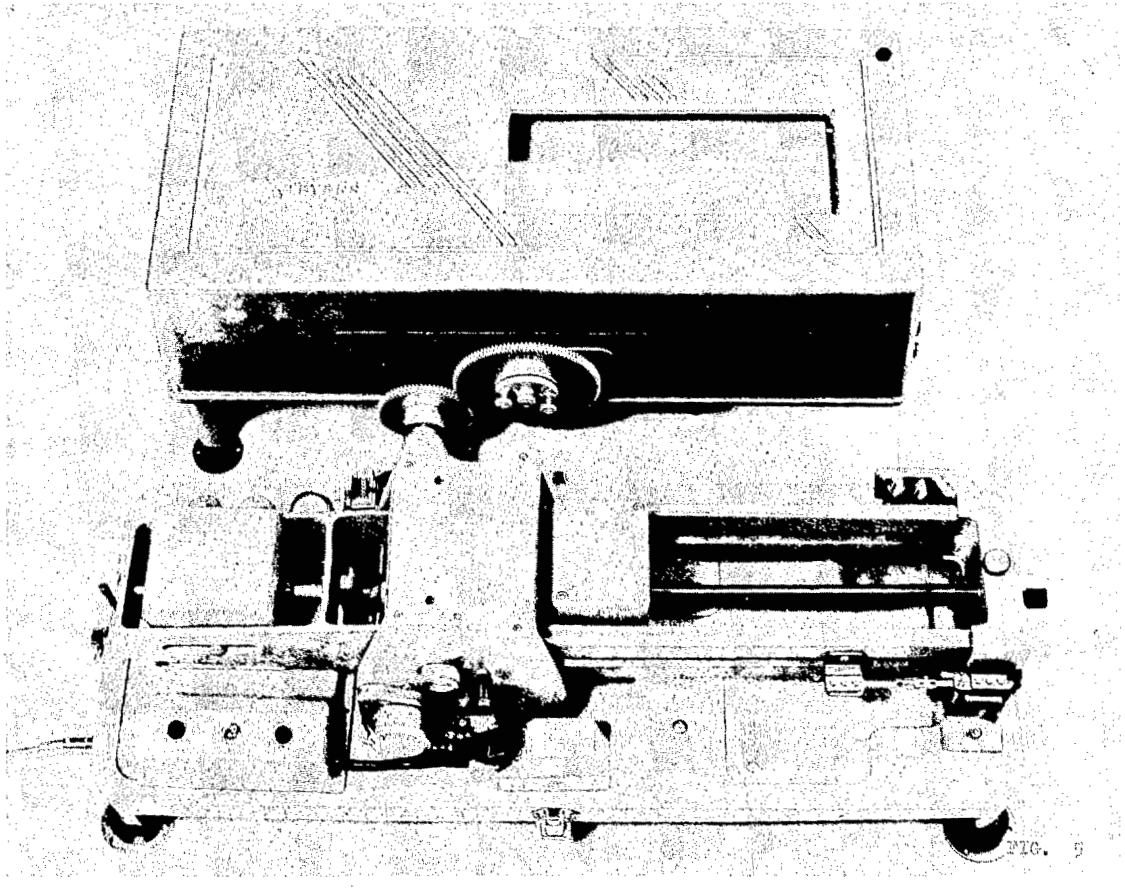


Figure 5. *A DP 30 shown operating a Stevens A35 long-term graphical recorder. Alternative connecting gears and chain drives are available*

Conclusions

The performance levels obtained by the Sherlock Pressure Sensing Unit (See Appendix) justifies the choice of the force-balance concept and there is every indication that the instrument will become universally accepted. Already some seventy units have been manufactured and are operating over all parts of Australia, New Guinea and Malaysia.

The instrument lends itself to further developments – higher ranges can be attained and a differential model is being considered. Space is available beneath the instrument cover for mounting accessory items, such as decoders and transmitters for telemetering systems and already such devices have been fitted.

APPENDIX – SCHEDULE OF ACHIEVED PERFORMANCES

Range: 0-70 feet (0-21.5 m).
Sensitivity: 0.005 foot (1.5 mm) – almost complete absence of a "stepped" trace.
Follow rate: 37 foot/minute (12.5 m/mm).
Torque output: $>$ 100 oz.in.
Electrical: Operates on 6 V dry battery for twelve months.
Accuracy: Better than $\pm$ 0.02 foot for 0 – 35 feet ($\pm$ 6 mm for 0 – 10.7 m) ;
Better than $\pm$ 0.10 foot for 35 – 70 feet ($\pm$ 3 cm for 10.7 m – 21.5 m).
Mechanical efficiency: High at least 200 per cent better than manometer-servo.
Reliability: Very good – very few reported instrument failures.
Temperature effects: None noticeable in ambient range.

II. GAS-ELECTRIC TRANSDUCER

by R. L. Hitchcock and H. L. Stark

The development by the Irrigation and Water Supply Commission, Queensland, of an instrument to meet the above parameters was commenced in the year 1956/57 (Mortley 1961). With progressive development, the gas-electric transducer, now used by the Commission as standard, has evolved to a reliable, accurate unit. Faults of some description or another occur in less than five per cent of units installed and have been traced to faulty microswitches in the recorder units, initial assembly of bubbler flow gauges, percentage manufacturing error per batch in transistor components and assembly of motor units. An occasional breakdown occurring due to the circuit design is described later.

Description of a basic installation

A typical layout of an installation is shown in Fig. 1. The equipment comprises:

1. Gas supply bottle (compressed CO_2) ;
2. Gas control board on which are mounted a manometer control valve, and an automatic adjustable bubble sight gauge; this is shown in Fig. 2 ;
3. Transducer instrument including manometer (mercury in stainless steel tube); case; winch motor, gearbox, drum, and contacts suspension cable; float contacts (up-down) remote sensing assembly; transistor (up-down) circuit card ;

4. Strip chart stream-level recorder mounted on top of transducer case ;
5. Gas bubbler tube, from control board to stream bubble outlet in stream ;
6. Dry batteries and cabling ;
7. Hut to house the instruments ;
8. Supporting structure for bubble outlet in stream.

Operation of a basic installation

The operating gas supply passes from the bottle through a pressure regulator and a pressure-reducing valve to the control board.

This control board (previously described) can be used to pass gas for normal operation, or for blow-through of the bubbler tube to the stream, and is therefore provided with the relevant outlets.

The gas supply for normal operation is connected through the control board to one leg (the pressure leg) of the mercury-stainless steel U-tube manometer, and at the same time to the inlet of the gas bubbler tube, which has its outlet in the stream.

The other leg (atmospheric leg) of the mercury U-tube manometer houses a remote sensing "up-down" float contact assembly at the mercury surface.

In normal operation, the gas back-pressure developed due to immersion of the bubbler outlet in the stream is a measure of immersion depth of the bubbler outlet and consequently of stream level. This is related to a surveyed R.L. of the bubbler outlet. The bubbler outlet may be either open as in Fig. 1 or enclosed in a double-graded pebble-filled filter enclosure when the chosen location makes it necessary to guard against blockage by natural random deposits of sand or silt.

The measured stream height and gas back-pressure thus causes a displacement of the mercury in the U-tube and a consequent vertical movement of the mercury surface in the open leg of the manometer. This action causes the following to occur in the float assembly: (a) closure of "up"-contact on stream level rising; or (b) closure of "down"-contact on stream level lowering; and (c) consequent operation of winch motor and float contacts assembly, either up or down, to open the contact and restore the float to a condition with both contacts open and the winch motor stopped.

The "up-down" float contacts are connected to a transistor and components network card by which appropriate rotation of the winch motor is controlled.

The winch motor drives the winch "up-down" drum through a gear system and drives the pen-shift mechanism of the proprietary strip chart level-recorder through a spring belt and gear system.

This motor, and that to move the strip chart, are powered by a dry battery system. A battery pack lasts approximately twelve months.

Normal consumption of CO_2 is approximately 20 cm^3 per minute at a bubble rate of 26 to 30 bubbles per ten seconds. At this rate a bottle content of 50 lb of CO_2 gas lasts approximately twelve months.

Accuracy. When used with a strip chart recorder, which incorporates a reversing pen system, the stream level is visually interpreted to one tenth of one foot (30,5 cm) at a scale of 1 inch (2,54 cm) equals 1 foot (30,5 cm). Manual subdivision of the tenths gives an increased accuracy. When used with a punched tape recorder. the stream level is punched to one-hundredth of one foot. This

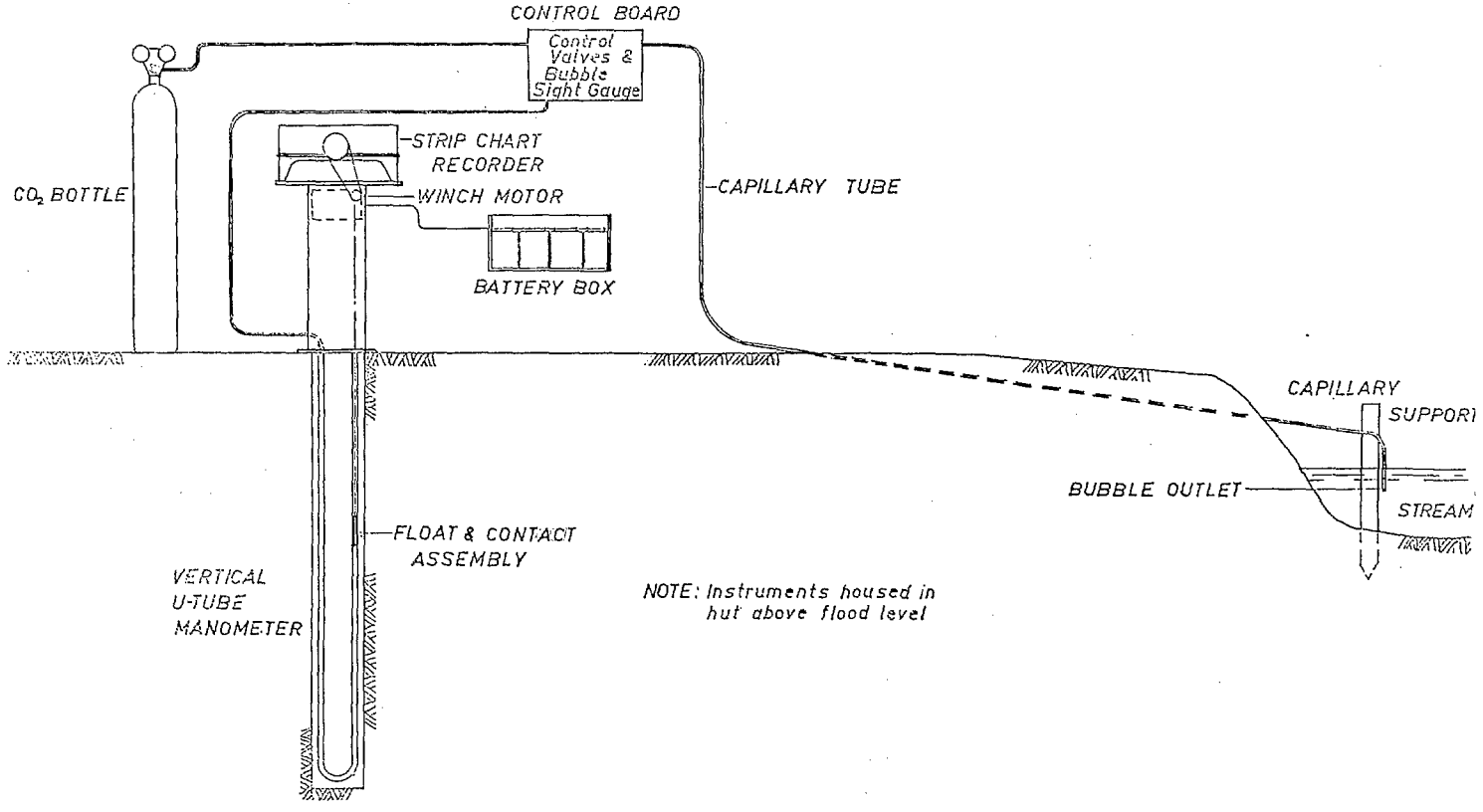


Figure 1. *Typical installation layout of a gas-electric transducer*

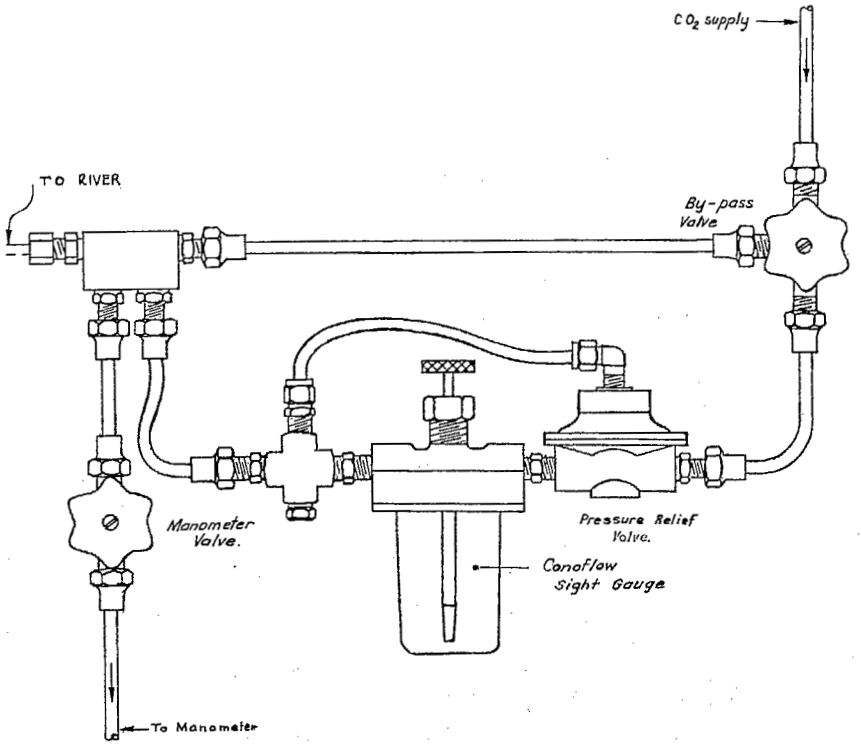


Figure 2. *Gas bubble control board*

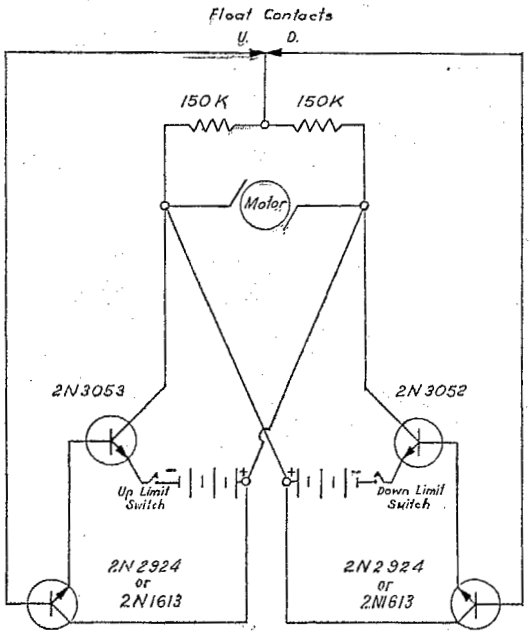


Figure 3. *Gas-electric transducer transistor switching circuit for float contact assembly*

latter type of recorder is being completely replaced by the strip chart recorder instrument owing to difficulty of maintaining the punch system in the field, inaccuracies of punching and consequent read-out machine problems, and the necessity to convert punched taped readings into a more easily interpreted form.

Development. Over the last years two major alternatives for the switching mechanism were developed and tested in the Commission's instrument laboratory. These are referred to for convenience as the (a) probe; and (b) float-contact.

The probe type consisted of two pointed metal conductors set in a resin block with their pointed ends set at slightly different levels. This was suspended in the manometer, and the probe points connected to a circuit such that there was equilibrium when one probe was in the mercury and the other just above it. A rise or fall in the mercury produced appropriate responses in the circuit so that the probe assembly was raised or lowered and at the same time the recorder was actuated by a corresponding amount. Various materials including stainless steel and platinum were tested for the probe points, the best being platinum wire with a 90° point carefully polished, but the design was abandoned mainly because of problems in maintaining the mercury surface clean enough to ensure reliable operation.

Various designs were also developed and tested for the float-contact system, but the standard design now adopted involves a fully transistorized circuit which is illustrated in Fig. 3. This system has proved very reliable, except for an occasional fault under conditions of extremely slow rate of stream rise, which occasionally produces overheating of transistors.

Installations. Since the inception of this programme, some 220 stations have been equipped with this instrument with ranges of 50, 75 and 100 feet (15, 22.5, 30 m). Over half this total number comprises the latest design which has now been standardized. It is of some interest that one of the original-type instruments has recently been replaced after some ten years of trouble-free operation.

Gas system. Experiments were carried out to determine gas flow rates and capillary lengths. The head on the capillary was varied by means of water and mercury columns. During this research, times to fill various capillary lengths at various heads were obtained, which gives an indication of time lags which may be experienced in the field on initial fill. These experiments also indicated that friction losses in the capillary tube were negligible.

Manometers. Originally these were manufactured from tubes of varying cross-section, horizontally welded together, which then exhibited interior obstructions due to slag, etc. They are now continuous U-tubes of uniform cross-section. Displacer rods can be fitted, if required, to vary the internal cross-section of the manometer tube.

Motors and gearboxes. The original instruments were designed with components obtained from disposals aero equipment, but instruments are now assembled using commercially available components.

Maintenance. Routine maintenance such as replacing gas bottles, batteries, winding recorder clocks, replacing charts, checking zero of chart against stream level and

replacing suspect transistor circuit cards is done by the hydrographic staff in their periodic two- or three-monthly trips.

A set of instructions is being prepared for their general use.

Conclusion

The general over-all design has been modified and simplified with time such that now a reliable, accurate, rugged, trouble-free, simple, easily maintained instrument has been patented in Australia by, and is in standard service with, the Irrigation and Water Supply Commission of Queensland, Australia.

An interesting side-note in conclusion however is that, in an endeavour to combat occasional vandalism, it has been found necessary in some of the installations to attach bullet-proof shields around the instrument and recorder, as bullets have been fired through both shed and instrument.

Acknowledgments

Acknowledgment is paid not only to various Government Officers, in particular Mr. H. R. Mortley and Mr. C. T. Leadbetter of this Commission, but also to those of various Departments of the University of Queensland and the Weapons Research Establishment, Salisbury, South Australia, for their work and thought given during the developmental stages.

The authors wish to thank Mr. F. B. Haigh, Commissioner of the Irrigation and Water Supply Commission, Queensland, for permission to prepare and publish this paper.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Mortley, H. R. 1961. Gas electric transducer. A device for transmission of river heights to the Leupold Stevens or other recorder. *Journal of the Institution of Engineers, Australia.* June.

Stream gauging network of the lower Mekong basin

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Abstract. The development of the water resources of the Mekong river and its tributaries is the principal purpose of the vast regional scheme called the Mekong Project. An orderly development programme for the utilization of the water resources of the basin requires accurate quantitative data on the characteristics of the river. The availability of such data will allow the permutation of plans for the regulation of the river for the benefit of those people living in the basin.

RESEAU HYDROLOGIQUE DU BASSIN INFERIEUR DU MEKONG

Résumé. Le développement des ressources hydrauliques du Mékong et de ses affluents est le but principal d'un vaste projet appelé le Projet du Mékong. Un programme rationnel de développement des ressources en eau du bassin requiert des données quantitatives précises. L'existence de ces données permettra l'établissement de plans de régularisation du fleuve pour le bénéfice de la population du bassin.

RED HIDROLÓGICA DE LA CUENCA INFERIOR DEL MEKONG

Resumen. El desarrollo de los recursos hidraúlicos del Mekong y de sus afluentes constituye el objetivo principal de un extenso proyecto llamado Proyecto del Mekong. Un programa racional de desarrollo de los recursos en aguas de dicha cuenca require unos datos cuantitativos detallados. La disponibilidad de estos datos permitirá el establecimiento de planes de regularización del río que benificiarán a los vecinos de la cuenca.

ГИДРОМЕТРИЧЕСКАЯ СЕТЬ НИЗОВЬЯ БАССЕЙИА МЕКОНГ

Резюме. Развитие водных ресурсов реки Меконг и его притоков является главной целью обширной программы называемой — Проект Меконга. Правильное осуществление программы с целью использования водных ресурсов басеейна требует точных количественных данных характеристик реки. Наличие таких данных позволит внести уточнения в планы регулирования реки на пользу населения, проживающего в этом бассейне.

GENERAL

For centuries, the Mekong river has affected the lives of millions of people living in its basin. Utilization of the river has largely been of its natural régime which is essentially unaffected by man. The Mekong river is one of the largest rivers and is approximately 2,600 miles (4,150 mm) long, with a drainage area of 307,000 square miles (800,000 km^2). The lower Mekong basin has a total recorded length of 1,900 miles (3,000 km). This lower Mekong river basin lies in the riparian countries of Cambodia, Laos, Thailand and Viet-Nam.

It has long been apparent that the Mekong river is a great natural resource for the people of the region. The riparian countries have established a Committee for Co-ordination of Investigations of the Lower Mekong River Basin for the purpose of promoting, supervising, controlling and co-ordinating the ultimate development of comprehensive plans for the river basin. The Co-ordinating Committee has obtained technical assistance from the United Nations and the over-all programme is supported by twenty-six contributing nations,[1] fourteen agencies of the United Nations,[2] twelve other organizations,[3] and eleven companies[4]. In order to conduct investigations for utilization of the water resources of the basin, accurate and long-term data on the characteristics of the river are required. These data will provide the basis for plans to regulate the river for the benefit of the people in the region. The information required for determining the characteristics of a river include data on stream-flow, precipitation, evaporation, quality of water and energy slope of the rivers.

The need for such data was strongly emphasized by the report of the United Nations Survey Mission dated 23 January 1958, usually referred to as the "Wheeler Report". The United States Government through the International Co-operation Administration undertook to implement the recommendations of the Wheeler Report regarding collection of hydrologic and survey data as its initial contribution to the Mekong Project.

STREAM GAUGING NETWORK

The Harza Engineering Company (United States) under contract from the International Co-operation Administration carried out the initial work of establishing a hydrological network in the lower Mekong basin in February 1959, and today there exist twenty-four stream gauging stations on the main stream of the Mekong river (200 gauging stations in the basin, including its tributaries in four riparian countries). Out of these twenty-four gauging stations, ten stations are operated by Thailand. The remainder are in Cambodia, Laos and Viet-Nam. Twenty of these stations have automatic stage recorders.

In measuring the discharge of the river, routine measurements are made at customary intervals at times of low and medium flow. But when high floods occur, observations are difficult due to inadequate equipment. To overcome this difficulty, the Mekong Committee with the help from the United States Government has obtained equipment for measuring discharge of large rivers enabling measurements to be taken in a short time even during the high floods. Measurement by this method is

1. Australia, Austria, Belgium, Canada, Republic of China, Denmark, Finland, France, Federal Republic of Germany, Hong Kong, India, Indonesia, Iran, Israel, Italy, Japan, Netherlands, New Zealand, Norway, Pakistan, Philippines, Sweden, Switzerland, United Arab Republic, United Kingdom, United States of America.

2. UNOTC, UNDP(SF), ECAFE, IAEA, IBRD, ILO, FAO, Unesco, UNICEF, UNIDO, WFP, WHO, WMO.

3. Asian Institute for Economic Development and Planning, Asian Highway Transport Technical Bureau, Advisory Board, Asian Development Bank, Asia Foundation, Ford Foundation, International Rice Research Institute, Resources for the Future Inc. Pan Pacific and South-East Asia Women's Association, Ramon Magsaysay Award Foundation, Rockefeller Foundation, New Zealand Freedom from Hunger Campaign.

4. Bulgar Associate Inc.
Dentsu Motion Picture Co.
Gestetner (Eastern) Ltd.
Price Waterhouse Co.
Shell Oil Co.
Imperial Chemical Industries.
Geteza Surveying Co.
Esso Oil Co.
Hindustan Zinc Co.
Nippon Electric Co.
Sycip. Gorres, Velayo and Co.

known as the "moving boat method"[5]. Six sets of the equipment were delivrered in 1966 to the four riparian countries.

The National Energy Authority of Thailand has set up a further twenty-eight stream gauging stations where discharge measurements are made and another thirty-three stations where only staff gauge readings are taken. These are on the tributaries of the Mekong river inside Thai Territory.

TYPE OF GAUGE

The twenty automatic gauge recorders on the main stream can be subdivided as follows: bubble gauge, 19 ; float gauge, 1.

It is interesting to note here that nineteen of the automatic level recorders are used on the mainstream and only one is of the float gauge type. Also the bubble gauge, from the moment it was produced, has been used on the Mekong even before it was adopted for use elsewhere, including the United States. The choice as to which type is preferable depends on many factors, but in developing nations, such as the countries in the Mekong region, these factors are different from those in the United States or Europe.

Most of the recorders obtained are supplied to the four riparian countries through the United Nations; the countries themselves were concerned only in the cost of installation and operation.

It is felt necessary to mention something to indicate the order of such costs. The installation cost of a float gauge on the mainstream of the Mekong is more than twenty times the cost of the instrument, while installing the bubble gauge would cost only about two times the cost of the instrument. To allocate a large sum of money for the installation of a level recorder is a big burden on a developing country. However, the cost of operation of a bubble gauge is many times that of a float gauge and the instrument itself is much more complicated than the float gauge.

The decision taken as to which gauge is preferable is obvious from the number of gauges in operation. On the tributaries however, more float gauges are used than on the mainstream, as the construction cost is lower.

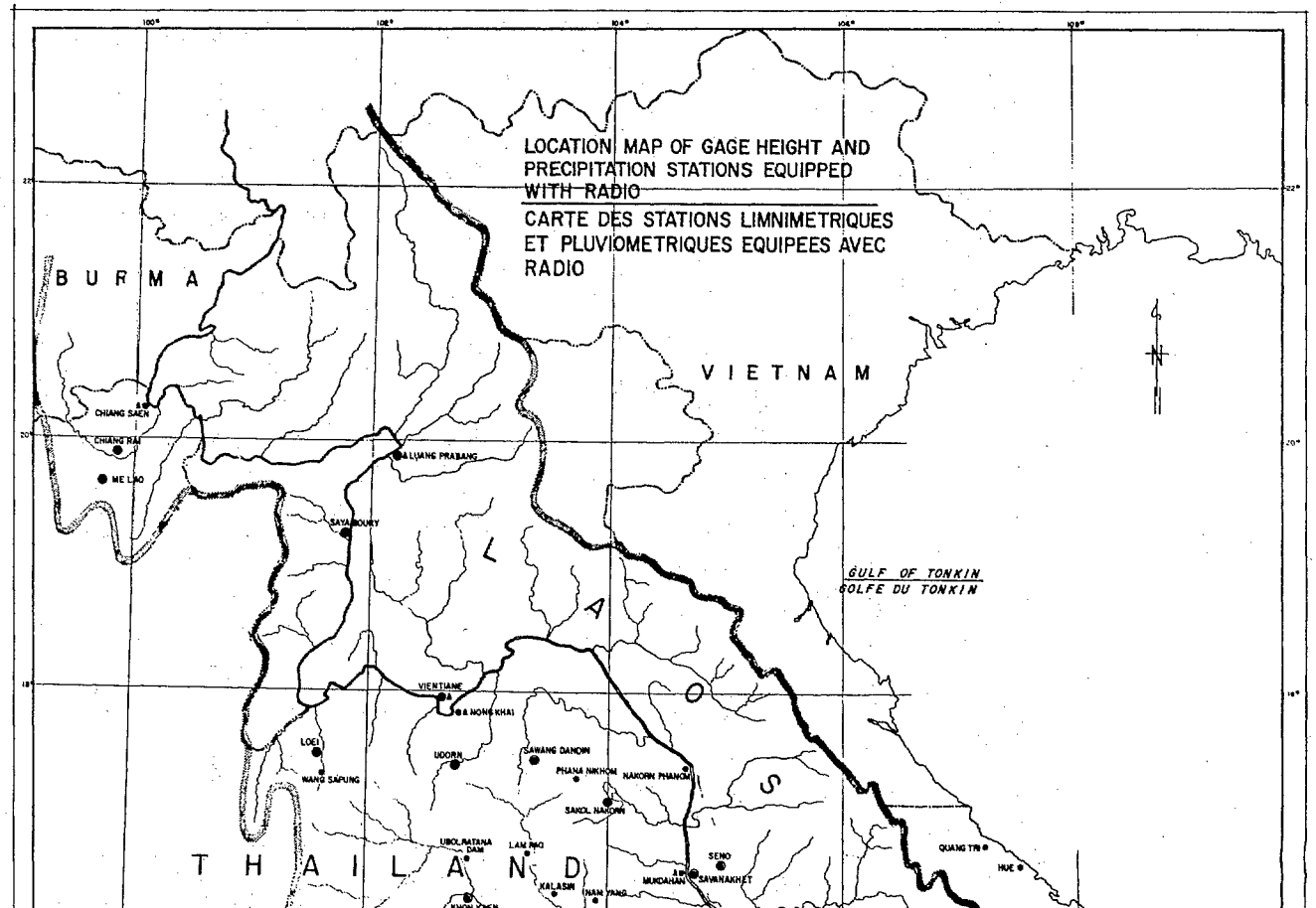
OPERATION AND MAINTENANCE

The operation cost of the bubble gauge is much higher than that of the float gauge. The obvious extra cost is the cost of the nitrogen gas that has to be used in the bubble gauge. Because the bubble gauge is more complicated in its construction, it requires a skilled person to supervise its operation. It has many other defects and thus requires more frequent inspection. Maintenance costs are also much higher.

DIFFICULTIES AND LIMITATIONS

The most serious difficulty faced at the present in expanding the stream gauging network to an adequate level is the failure by many officials concerned to recognize the importance of hydrometeorological data. There is insufficient awareness of the

5. Measurement of discharge by the moving-boat-method, by G. F. Smoot and C. E. Novek. USGS Techniques of Water Resources Investigations Book 3, 1968, Open-File.


LOCATION MAP OF GAGE HEIGHT AND PRECIPITATION STATIONS EQUIPPED WITH RADIO
CARTE DES STATIONS LIMNIMETRIQUES ET PLUVIOMETRIQUES EQUIPEES AVEC RADIO
BURMA
VIETNAM
LAOS
THAILAND
GULF OF TONKIN
GOLFE DU TONKIN
CHIANG SAEN
CHIANG RAI
ME LAO
LUANG PRABANG
SAYABOURY
VIENTIANE
NONG KHAI
LOEI
UDORN
SAWANG DANDIN
PHANA NIKHOM
NAKORN PHANOM
WANG SAPUNG
SAKOL NAKORN
UBOL RATANA DAM
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KALASIN
NAM YANG
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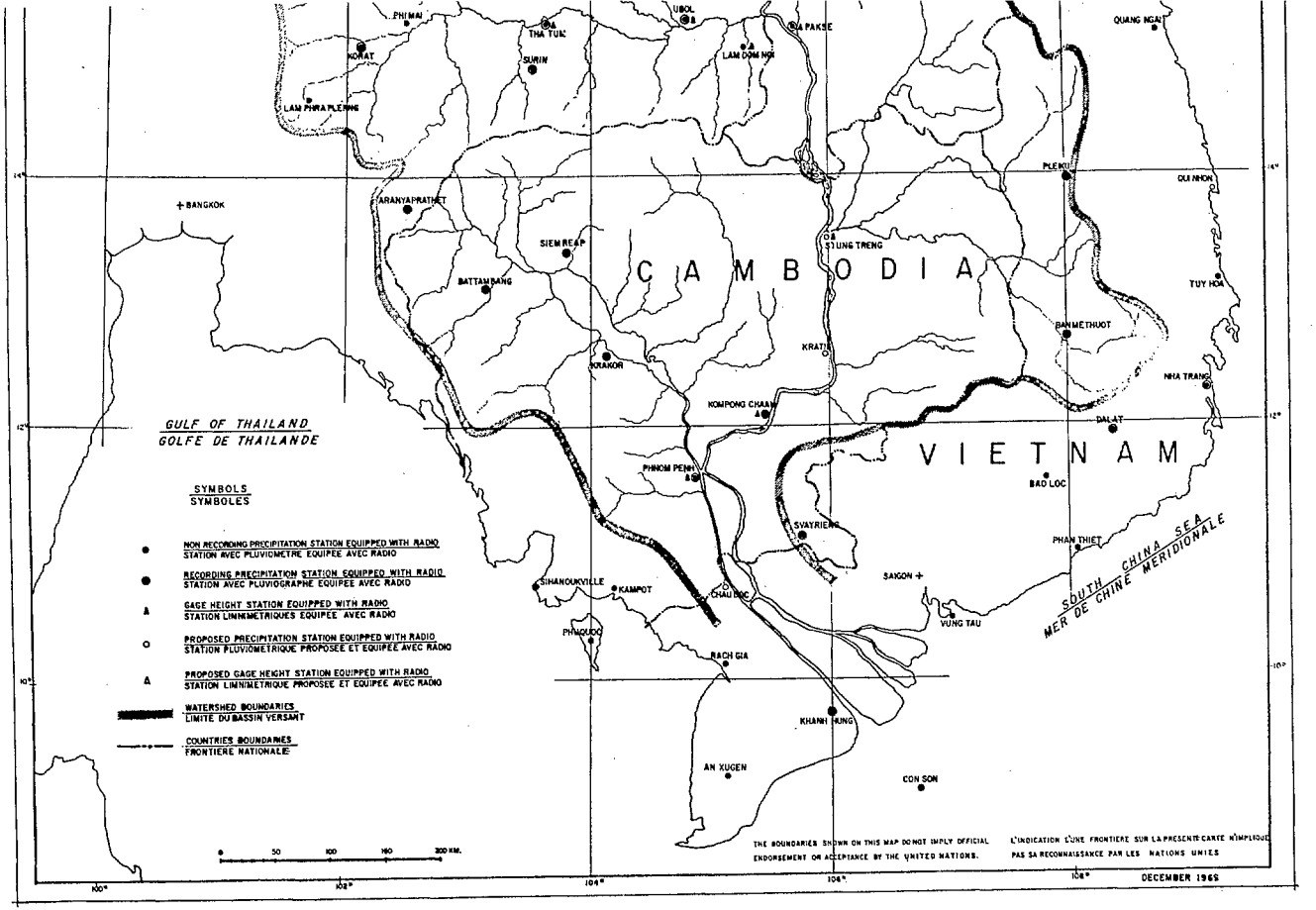


Figure 1. *Location map of gauge height and precipitation stations equipped with radio*

loss which results from over-design and the damage which results from under-design of various structures. Moreover, inadequate amounts of funds are allocated for the hydrological network as the data are not used immediately and the collection process is very time-consuming. It requires a period of from five to ten years to obtain the data necessary for design of small projects. Thus the necessity to expand the network is not very obvious to persons not directly related to the field.

In addition, the problem faced by the riparian countries in running an efficient network is the difficulty in training personnel for the collection of these data and in making them realize their importance. On many occasions data have to be discarded because they are not reliable. Lack of funds does not allow the payment of higher wages. Lack of equipment is also another major setback.

Attempts are being made to overcome these difficulties by sending personnel overseas to receive training in this field and efforts are also being made to have adequate equipment and funds. This process will take time, however, especially in a developing country.

Water-level gauging by pressure measuring

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Abstract. The teletransmission or the mechanical processing of gauge heights requires additional work for the transformation of these data which are usually in the form of chart records. The punched-tape gauge produces data which can be electronically processed but the data are hardly decipherable, if there is no simultaneous analogue record. Parallel connexions of tape puncher, analogue recorder, teleophone recorder and installations for teletransmission need such a volume of equipment that it seems necessary to look for a new method of measurement which produces an electrical output.

A new device is described which, like the pneumatic or the hydrostatic pressure gauges, has a minimum of frictional losses or gear backlashes and which produces a directly usable output. This can be used, without difficulty, for such purposes as indicating, recording and teletransmission.

MESURE DES NIVEAUX D'EAU PAR MESURE DE LA PRESSION

Résumé. Télétransmission ou traitement mécanique des mesures de niveaux exige un travail additionnel pour transformer ces données enregistrées habituellement sous forme de graphique.

Le limnigraphe à bande perforée fournit des données aptes à un traitement électronique, mais très peu lisibles pour l'opérateur si on ne prévoit pas un enregistrement analogique simultané. Le montage en parallèle d'imprimeurs à bandes perforées, l'enregistrement analogique, les répliqueurs d'appel et les installations pour la télétransmission exigent un tel équipement que le besoin se manifeste de chercher une nouvelle méthode de mesure fournissant immédiatement des valeurs électriques et permettant de renoncer à la transmission mécanique des niveaux d'eau (par exemple, par flotteurs).

L'étude décrit un nouvel appareil qui, telles les jauges à pression pneumatique ou hydrostatique, présente un minimum de pertes dues aux frictions ou au jeu mort et livre immédiatement des valeurs utilisables. Ces valeurs électroniques peuvent être exploitées sans difficulté à d'autres fins (indication, enregistrement, télétransmission, etc.).

MEDIDA DE NIVELES POR PRESIÓN

Resumen. La teletransmisión, o el tratamiento posterior mecánico de las alturas de nivel, exige inversiones adicionales para transformar estos datos que hasta la fecha se han registrado en forma de curva.

El registrador de nivel de cinta perforada obtiene, efectivamente, datos calificados para el ulterior tratamiento mecánico, pero que son difícilmente legibles para el encargado de servicio si no se prevé un registro análogo simultáneo. El montaje en paralelo de impresores de cinta perforada, registrador análogo, registrador telefónico, e instalaciones para la teletransmisión, exige una electrificación tan voluminosa que parece evidente la necesidad de buscar un nuevo método de medida que suministre de inmediato valores eléctricos y que permita renunciar a la transmisión mecánica de los niveles de agua (p. ej., por flotadores).

Para estos fines han aparecido nuevos sistemas, como la medición de presión neumática e hidrostática, que con un mínimo de pérdida debida a la fricción o juego muerto en el engranaje dan en forma inmediata valores eléctricos. Éstos se pueden utilizar sin dificultad para otros fines, tales como indicación, registro, teletransmisión, etc.

Completado con fotografías y gráficos, en este informe se da una idea de la utilización práctica de dichos equipos.

ИЗМЕРЕНИЕ УРОВНЯ ВОДЫ ПУТЕМ ИЗМЕРЕНИЯ ДАВЛЕНИЯ

Резюме. Передача на расстояние или механическая обработка измеряемых высот требует дополнительных операций по преобразованию этих данных, которые до сих пор в большинстве случаев вычерчиваются как аналоговые кривые.

Регистратор, использующий перфоленту, дает данные, пригодные для электронной обработки, но эти данные почти не доступны для человека, если одновременно не производится аналоговая или расшифрованная запись. Параллельные связи перфоратора, аналогового устройства, телефонного регистратора и исследования для телепередачи зависят от столь обширной электрификации, что целесообразнее найти новый метод измерений, дающий непосредственно электрические значения и не требующий механической передачи уровней (например, с помощью поплавков).

Для этого предложены новые пневматические или гидростатические измерители давления, которые при минимальных потерях на трение или передаточного механизма дадут непосредственно электрические данные. Их легко можно использовать для таких целей, как указание, запись, передача сведений об уровне и т. д. С помощью фотографий и графиков доклад дает представление о практической работе такого оборудования.

INTRODUCTION

For many years there have been attempts to improve the traditional methods of water-level gauging, which usually rely on staff gauges and float recorders. On wide rivers or streams with considerable changes of water-level, float gauges require expensive structures with deep gauge wells and long intake pipes. The maintenance costs of these structures can be considerable. Pneumatic or air-pressure gauges should overcome this disadvantage but they too need long air-pressure pipes in some cases which also give rise to difficulties.

The desire is often expressed that records from gauges should be obtained by telephone or radio. Data could be transmitted instantly to a computer, thus eliminating the processing that is usually required. So the question was raised as to whether it was possible to find a new method of measurement which could be converted directly into electrical impulses. This condition is met by the "Druckmessgeber" (perhaps one could translate this expression as "a pressure transmitting gauge") which was developed in West Germany and has been used successfully for several years at our gauging stations.

DESCRIPTION OF THE MEASURING EQUIPMENT (See Fig. 1)

An air-tight spring hose having a concertina shape (2), is fastened with its open upper end in a stable protective pipe (1). Its lower end is closed, and is connected to a tension spring (3) which is fastened to a bridge in the mouth of the protective pipe. In the normal position, both the tension spring and the spring hose are lightly prestressed.

Above the protective pipe is an air-tight chamber (4). This connects with the interior of the spring hose (2) below, and also with the inside of a balloon (7) above, but is completely shut off from the atmosphere. Condensation is prevented by special additives. The chamber encloses a circular variable resistance (5) which transforms the deformation of the spring hose into a resistance value. This value is

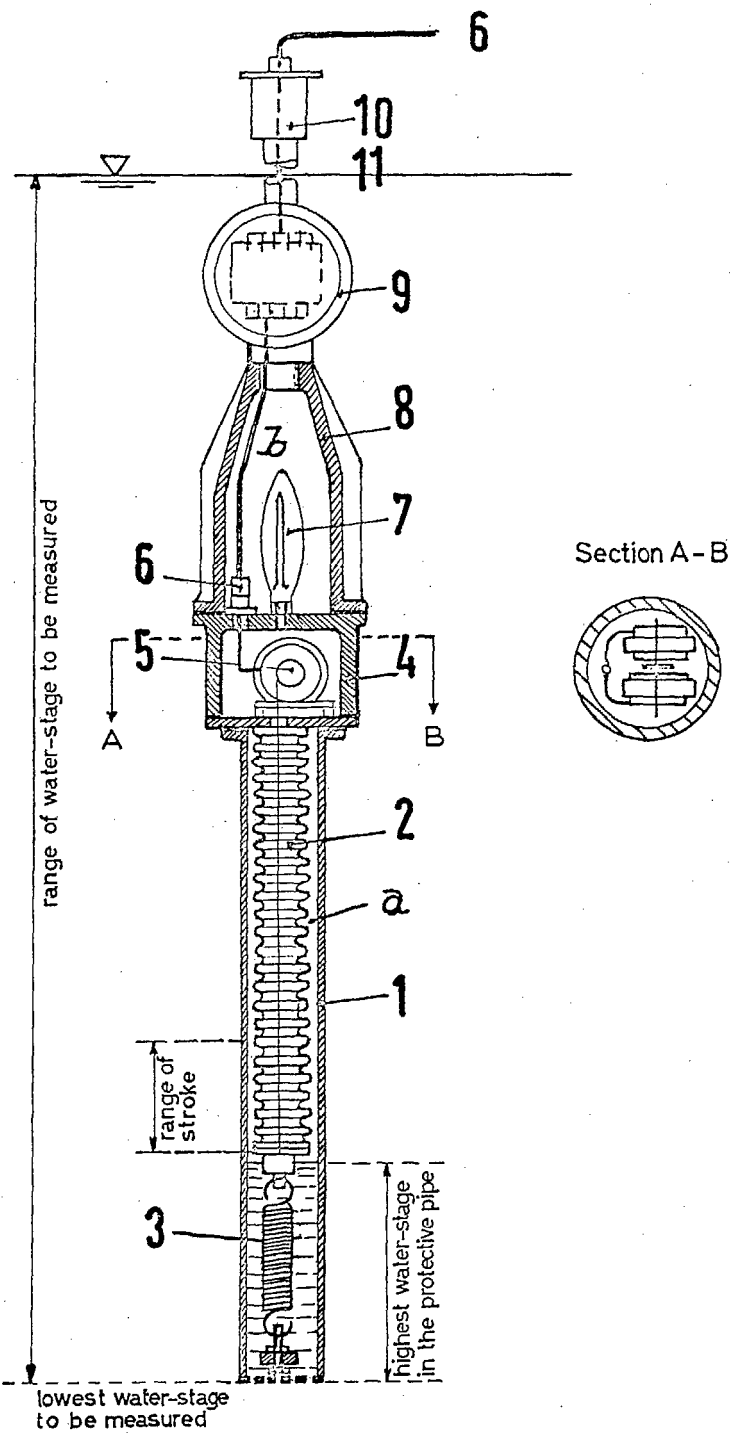


Figure 1. *Measuring equipment*

transmitted by the measuring cable (6) through the cover (8) to the contact box (9). The passage between the chamber (4) and the cover must be secured and made air-tight. The inside of the cover and the contact box are connected to the atmosphere by a flexible tube, the vent of which is located above the highest water level. In this way the air pressure inside the cover is always kept at the atmospheric level. For the sake of convenience, the measuring cable (6) may be led through the air-vent pipe to the recorder house or to the transmitting station.

MODE OF OPERATION

The system works according to the hydrostatic principle that the pressure in a liquid increases in proportion to its depth. If the instrument is plunged into a liquid, the air pressure in the interior (a) of the protective pipe (1), which is cut off from the atmosphere, increases according to the depth. This causes an external pressure on the spring hose which compresses it in the longitudinal axis. However, the range of movement of the spring hose (2) and the tension spring (3) are limited, and the system is so calibrated that the rise of water-level within the protective pipe with increasing depth will never reach the spring hose. As an additional security against silting and the colonization of plants and animals which perhaps could influence the measurements, the spring hose and tension spring are made of corrosion-resistant material.

A draw-string inside the spring hose has one end fastened to the bottom of the hose and its upper end wound round the spindle of a variable resistance (5). This spindle and the draw-string are also lightly prestressed by a spiral spring. The deformation of the spring hose is transformed by the draw-string into a rotation of the spindle which is connected to a sliding contact on the variable resistance. In this way, a resistance value is obtained which can be transmitted through the cable (6) for a digital display, registration, teletransmission and so on.

The expansible balloon (7) in the cover (8) takes up the surplus air expelled from the spring hose chamber (2).

On account of its special structure, this design works with extraordinarily small frictional losses. Only the sliding contact and the bearing bush of the potentiometer spindle involve friction. Therefore the accuracy of the measurements is high.

According to the range of measurement, different sizes of spring hoses may be used. The maximum measurable range of water-level fluctuation for this type of gauge is up to 200 metres. Naturally the precision depends on the amplitude to be measured. For up to 10 metre water-level fluctuations, an accuracy of 1 cm can be obtained. In measuring smaller amplitudes, the precision can be improved to several millimetres. However, such extremely accurate measurements are not important in practice, owing to rapid disturbances in the water-level and the inevitable inaccuracies of the rest of the equipment. In the case of considerable water-level fluctuations, one could improve the absolute precision by using several small-range instruments in series.

Water-level measurements obtained in this way may be processed in code or in other ways. If a mains supply of electricity is not available, the gauge can be operated with batteries. To extend the life of the batteries one should try to provide at least a part of the required energy in another way (for instance the paper-chart used for recording could be operated by clockwork). For emergency operation, a set of batteries should be kept available to provide all the necessary power for several hours.

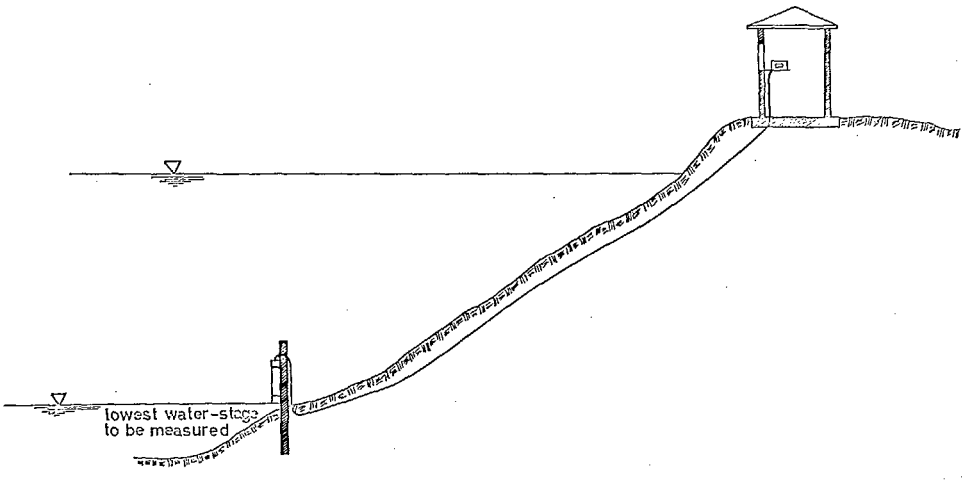


Figure 2. *Arrangement of measuring devices*

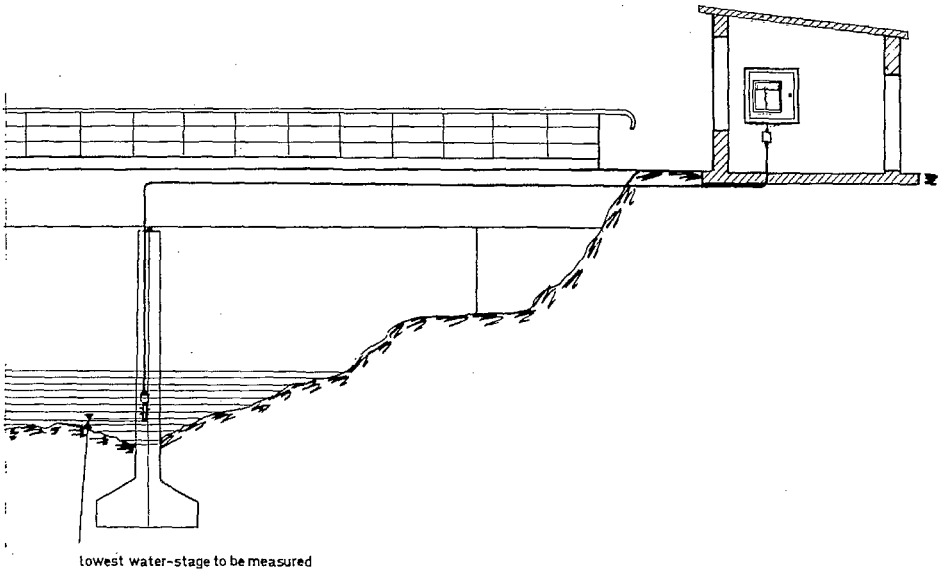


Figure 3. *Another arrangement of measuring devices*

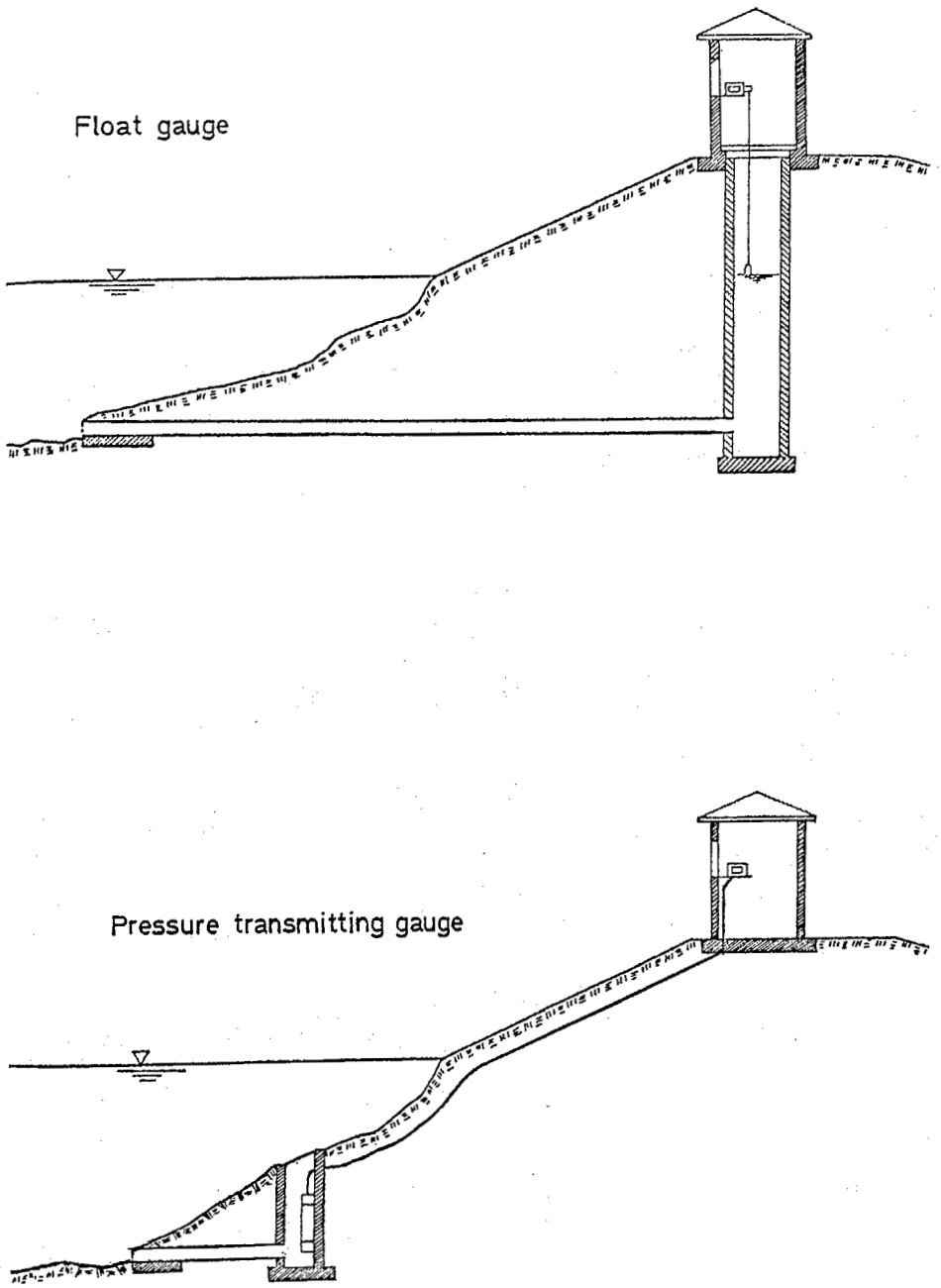


Figure 4. *Comparison between float gauge and pressure transmitting gauge*

A protection against lightning has proved to be necessary and is installed in the contact box (9).

EXAMPLES OF PRACTICAL APPLICATIONS

One advantage of the pressure transmitting gauge is the considerable reduction in installation costs and in the time of construction, particularly for small streams. In very simple cases, the instrument could be buried in the river bed. Rough stones could be used as a protective covering for leading water to the system. Naturally this procedure would not be practicable if silting might prevent the transmission of the pressure.

One could also attach the instrument ot a pole in the river, perhaps with an additional pipe to protect against abrasion by the bed-load. It could also be placed beyond the zone of bed-load movement in the river bed but below the lowest water-level, which must be measured (Fig. 2). Such a means of installation is practicable on bridge-piles (on the downstream side or in grooves on the side-walls). Only the cable has then to be led to the recorder-house (Fig. 3). The influence of the bridge piles on the velocity of flow and on the water-stage can be compensated for.

Where these possibilities do not exist and where bed-load transportation or floating material do not allow this method of installation, an intake and well can be made at a comparatively small expense. Needing very little maintenance and being adjusted electrically in the recorder-house, the instrument can be employed in a very simple well. Only the cable within the flexible tube is led to the recorder-house (Fig. 4). Its simplicity of installation makes this instrument very suitable as a transportable gauge for experimental work or for temporary river studies.

In Northrhine-Westfalia this type of gauge has been tested with success for more than five years. Even where the surface of the protective pipes was covered by algae and animal-like organisms, the inside remained free from growths. Furthermore, weeds or floating material after underwater cutting have not influenced the accuracy of measurement.

DISCUSSION

Sherlock: You mentioned a range of 200 metres in your paper, and even a range of 600 metres. Does this represent the range of a standard instrument?

Zayc: It is possible to build structures which can measure up to 200 metres with one arrangement. In this event the errors are relatively high. An error of only 0.1 per cent in a distance of 200 metres will lead to a deviation of 20 cm. This is far too much. If you want to measure such a great range, several instruments with a range of 20 or 10 metres should be used.

Oebius: You mentioned an accuracy of 1 cm and that this deviation is very little, since the attached electronics would rule out the mistakes. However, taking into account the importance of a difference of 1 cm for the calculation of the water contents in a reservoir and taking into account the possibility that differences and mistakes may multiply themselves, greater accuracy seems to be necessary.

There are already sufficient devices with that accuracy and I do not see any advantage. It is necessary that the basic device itself avoids all possibilities of mistakes.

II Measurement of discharge
Jaugeage du débit

II.1 Measurement of discharge by measuring velocity in a cross-section

Jaugeage du débit par la mesure de la vitesse dans une section transversale

The analysis of the possibilities of current meter operation in turbulent streams

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Abstract. The present paper reviews the different current meters in operation in hydrometric networks. Special attention is paid to the analysis of the influence of turbulent flow upon the readings of current meters under different working conditions (rod or cable suspension).

Some recommendations are given concerning the application of current meters, depending on the hydrodynamic properties of the watercourses and operational practice; probable errors for different types of current meters are determined.

ANALYSE DES POSSIBILITES DES MOULINETS OPERANT DANS DES ECOULEMENTS TURBULENTS

Résumé. Le présent article passe en revue les différents moulinets utilisés dans un réseau hydrométrique, du point de vue de leurs caractéristiques opérationnelles.

Une attention particulière a été portée à l'analyse de l'influence de l'écoulement turbulent sur les indications données par les moulinets, compte tenu des conditions de travail (perche ou câble de suspension).

Des recommandations sont faites en ce qui concerne l'utilisation des moulinets ; elles s'appuient sur les propriétés hydrométriques des écoulements et les modes opérationnels ; les erreurs probables des différents types de moulinets ont été indiquées.

ANALYSIS DE LAS POSIBILIDADES DE FUNCIONAMIENTO DE MEDIDORES EN CORRIENTES TURBULENTAS

Resumen. El presente trabajo pasa revista a los medidores de corriente utilizados en la red hidrométrica desde el punto devista de sus caractérísticas de funcionamiento. Se presta especial atención al análisis de la influencia del flujo turbulento en las lecturas de los medidores de corriente con respecto a las condiciones de trabajo (suspensión por varilla o cable).

Se dan algunas recomendaciones relativas a la aplicación de los medidores de corriente en función de las propiedades hidrodinámicas de los cursos de agua y de la práctica de trabajo y se determinan los probables errores para diferentes tipos de medidores.

АНАЛИЗ ВОЗМОЖНОСТИ ИСПОЛЬЗОВАНИЯ ГИДРОМЕТРИЧЕСКИХ ВЕРТУШЕК ДЛЯ РАБОТЫ В ТУРБУЛЕНТНЫХ ПОТОКАХ

Резоме. В докладе дается обзор эксплуатируемых на сети станций типов вертушек с точки зрения их эксплуатационных характеристик.

Особое значение придается анализу влияния турбулентного потока на показания гидрометрических вертушек с учетом условий их эксплуатации (со штанги, троса).

Даются рекомендации по применению вертушек в зависимости от гидродинамических характеристик водного объекта и условий эксплуатации, а также вероятные погрешности для каждого типа вертушек.

At present various types of current meters are used in hydrometric practice; they differ not only in their design, but also in operational characteristics of propellers such as the initial velocity, critical velocity[1] componentness,[2] sensitivity to pulsation, etc. The great variety of current meter characteristics and the lack of a criterion for assessing the quality of current meters are unfavourable to the accuracy of streamflow measurements; it was impossible until recently to estimate the influence of flow turbulence on the records of a current meter, which in its turn led to the use of various types of current meters irrespective both of their operational characteristics and of the hydrodynamic properties of the water body.

As a result of the use of current meters sensitive to pulsation and to oblique currents, errors sometimes reached high values (up to ± 20 per cent) in the calibration of hydraulic structures and during measurements of turbulent flow.

The analysis of published reports on the fluctuation of velocity in rivers and in hydraulic structures and studies of the influence of pulsation on the accuracy of discharge measurements showed that the importance of choice of instruments was underestimated. Non-component[2] current meters were used as well as current meters having large moments of inertia; the influence of pulsation is exerted through the moment of inertia of the instrument.

If measurements are made by a horizontal axis current meter attached to a wading rod, the obliquity of the current tends to cause an underestimation of velocity, while the pulsation of flow results in overestimation of velocity.

No attention is paid either to certain design features of current meters such as the horizontal or vertical position of the axis of the propeller.

In the case of current meters with a horizontal axis attached to a wading rod one can expect that the error of the readings might be reduced by cancellation of the errors caused by pulsation and by the obliquity of the current, but in the case of current meters with a vertical axis the errors due to pulsation and obliquity of the current would be of the same sign (overestimation of velocity) and may reach 40 per cent under unfavourable conditions. This type of current meter is, on the whole, useless in a turbulent stream, especially for the calibration of hydraulic structures.

This casts some doubt on the recommendation, in the international rules for testing hydraulic turbines, concerning the use of current meters with propellers of diameter not less than 100 mm for calibrating hydro-electric plants. This recommendation contradicts the well-known fact that the moment of inertia of the working parts influences the instrument's readings in a pulsating flow.

Rating tanks usually allow current meter ratings to be performed at a speed not exceeding 3 m/sec. In hydrometric practice, however, current meters have to be used at velocities considerably exceeding the values indicated in the rating certificate. In such cases linear extrapolation of the rating curve is made to velocities, from two to three times above the upper limit of rating.

Investigations at the State Hydrological Institute showed that the beginning of the deviation from the rating curve (critical velocity) depends both on the parameters of the rating basin (width, depth) and on the parameters of the propeller (diameter, geometric pitch, disc ratio). When current meters are rated in tanks with an unlimited cross-section at speeds over 5 m/sec, and for some types of current meters even at speeds of 2.5-3 m/sec, deviation of rating curves from a straight line on an apparent

1. The initial velocity is the minimum velocity at which the rotation of the propeller of the current meter starts; the critical velocity is the maximum velocity beyond which the rating curve $v = f(n)$ is non-linear.

2. In this paper, component current meters are understood to be able to measure the projection of the stream velocity to the axis of the current meter.

reduction of velocity is observed, i.e. the error has a systematic character and may reach 3-5 per cent. Therefore, in extrapolation to higher velocities, the value of the critical velocity for each type of current meter should be considered. Operation of current meters is possible only up to their critical velocities.

All this required the analysis of the existing current meters to determine their operational characteristics, especially those such as their sensitivity to pulsation and componentness.

Since the influence of pulsation on current meter readings is especially important, the problem of determination of pulsation sensitivity of current meters is treated more thoroughly.

The necessary criteria which determine whether pulsation influences current meter readings are the moment of inertia of the working part and the sliding of the propeller. The absence of one of these conditions practically eliminates the influence of flow pulsation upon the meter readings.

Now the term "sliding" of the propeller or of the working part of the instrument will be analysed.

The principle of operation of a current meter is based on the relationship between the velocity of the water and the resulting angular velocity of the meter propeller.

For an ideal current meter operating in an ideal liquid, the relation

$$v = H_g \ , \ n_t \tag{1}$$

is valid where H_g is the geometric pitch of the propeller, n_t is the theoretical number of the propeller revolutions per unit of time.

In the above case the velocity of propeller movement should be exactly equal to the velocity of the liquid and an energy exchange must not take place. In order to obtain some energy from the flow to overcome the resistance to movement, the propeller should lag behind the flow velocity, i.e. the actual number of propeller revolutions should be a little less than the theoretical number of revolutions in equation 1. For the rotation of the propeller the equation

$$n_t - n_{ob} = S_a > 0 \tag{2}$$

must be valid, where n_{ob} is the actual or observed number of revolutions, S_a is the absolute propeller sliding.

It is possible to conclude from the above that the sliding of the propeller is characteristic of any type of current meter.

The analysis of the relative values of sliding, calculated for various current meters from their ratings while the current meter axis was coincident with the rating channel axis ($\alpha = 0^\circ$) showed that for a majority of meters the values of sliding are considerable only at low velocities, i.e. in the curved part of the graph $v = f(n)$. As the speed of current meter movement in the tank increases the value of the sliding sharply diminishes; for some types of current meter the value of sliding is relatively small even at a speed of 0.3 m/sec.

The rate of decrease of sliding with increase of velocity is different for various types of current meters. However the general feature is that at velocities less than the initial velocity the sliding equals 100 per cent. At velocities exceeding 0.3 m/sec, where relation $v = f(n)$ approximates a straight line, the value of the sliding becomes constant and small (from $-$ 2 per cent to $+$ 8 per cent).

At first sight this situation does not cause misgivings that errors might appear in velocity measurements due to pulsation, since their influence might be expected only

at low velocities corresponding to the curvilinear part of the rating curve. But, as will be shown, this statement is valid only for the operation of current meters in streams without oblique currents.

In the case of oblique currents, when current meters are operated from a rod, the sliding increases with obliquity (Fig. 1). In such a case the moment of resistance forces sharply increases due to the radial component affecting the axis of the current meter (radial pressure).

As is clear from the graph (Fig. 1) the sliding in oblique currents is considerable in the linear part of the rating curve as well. When the obliquity of the current equals 40° sliding might reach 30-35 per cent; it is therefore reasonable to expect an influence of pulsation over the whole range of velocities.

These arguments are true if the current meter is fixed firmly to the wading rod. When the meter is suspended from a cable the influence of pulsation is possible in the curvilinear part of the graph only, since in this case the current meter is turned along the velocity vector; in this position sliding of the propeller occurs at low velocities only and no errors due to pulsation are observed in practice.

As a rule superimposed vertical velocity curves for a turbulent stream, obtained by current meters attached to wading rods and cables, are plotted to examine the tendency of current meters either to underestimate or overestimate velocities due to oblique currents (Fig. 2). After correction of vertical-velocity curves for obliquity of current, they usually coincide. This is true only if there is no pulsation in the stream.

In hydrometric practice, one has most often to deal with turbulent streams characterized by pulsation both of velocity and of current direction. Under unfavourable conditions when the influence of pulsation dominates the influence of current obliquity, vertical-velocity curves, obtained by current meters operated from a cable and a rod, may present an inverse picture. The correction of the vertical-velocity curve for the cosine of the angle, if the curve is obtained by a cable-suspended current meter, will give a velocity distribution down the vertical similar to the actual distribution, whereas the curve corresponding to the measurements by rod-attached meter will indicate overestimated velocity values. The displacement of velocity curves, shown in Fig. 3, can be explained by the fact that, while measuring velocity from a wading rod, values are overestimated due to the influence of flow pulsation. As was mentioned above, one of the main conditions necessary for the pulsation to influence current meter readings is the moment of inertia of the working part of the meter.

Measurements show that the moments of inertia of the working parts of various meters differ substantially. Thus, for small current meters of GR-11 and GR-55 type, the value of the moment of inertia is within 0.1-0.2 g . cm . sec^2, while for current meters with vertical axes, such as the Bakhirev and Price current meters, the value is over 3 g. cm . sec^2. Even for current meters of the same type fluctuations of the values of the moment of inertia sometimes reach 90 per cent.

Such discrepancies in the moment of inertia for current meters of the same type, as well as for different types, cannot guarantee the stability of one of the most important characteristics of current meters – their sensitivity to pulsation.

Component properties of current meters were determined by direct testing. It is known that componentness is characterized by the ability of the meter to measure the projection of the stream velocity relative to the current meter axis. The componentness of the propeller is determined by the formula

$$n\,[\%] = \frac{n_0 \cdot \cos\alpha - n_\alpha}{n_0 \cdot \cos\alpha} \cdot 100$$

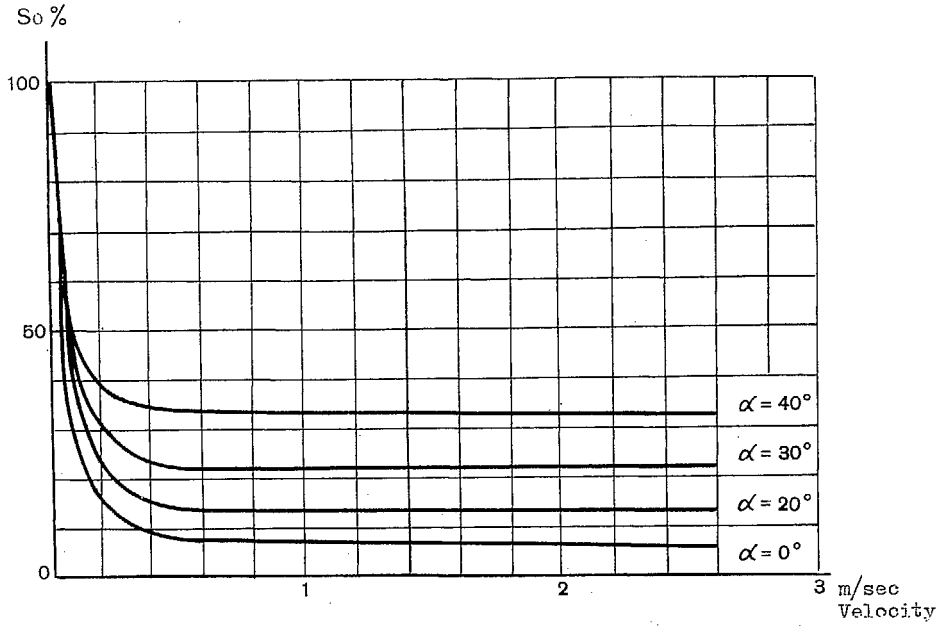


Figure 1. *Relation between relative sliding S_0 per cent of the propeller of the GR-55 current meter and the velocity of flow at flow angle α equal to 0°, 20°, 30° and 40°.*

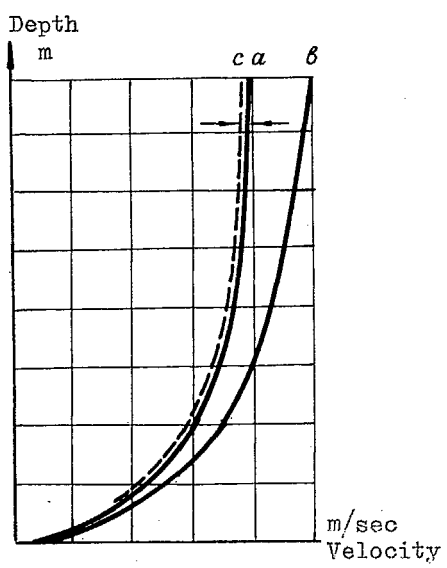


Figure 2. *Vertical-velocity curves:*
(a) Current meter attached to a rod;
(b) Cable-suspended meter;
(c) Cable-suspended meter, taking into account the angle of flow

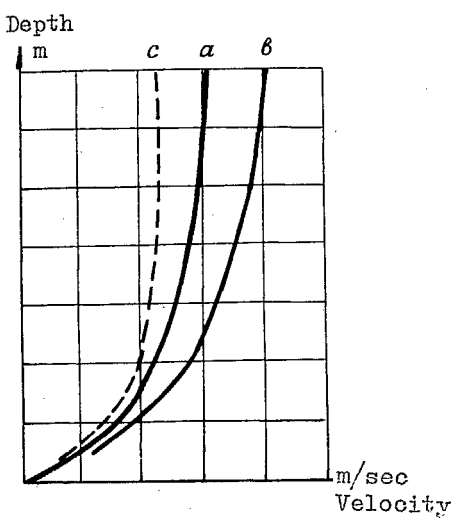


Figure 3. *Vertical-velocity curves in a pulsating flow:*
(a) Current meter attached to a rod;
(b) Cable-suspended meter;
(c) Cable-suspended meter, taking into account the angle of flow

where n_o is the number of revolutions of the propeller per unit of time at the normal position of the meter relative to the cross-section of the rating basin; n_α is the number of propeller revolutions per unit time at the same rating velocity, but with the current meter axis at an angle α to the basin axis.

Long-term investigations of component properties of current meters of various types showed that there are considerable differences of componentness of propellers both of various types and of the same type.

Propellers designed for optimum correlation of their parameters (GR-21, GR-55, GR-11-M current meters) have as a rule a componentness of the order of 1-5 per cent.

The analysis of the above characteristics of current meter propellers makes it possible to give some recommendations on the use of certain types of current meters relative to the hydraulic régime of the water body and operating conditions.

CONCLUSION

Current meters with vertical axes cannot be recommended for measurements in a turbulent stream; meters with characteristics similar to GR-21, GR-21M, GR-55 and GR-11M types should be recommended for use on network stations.

Small-size current meters similar to GR-55 and GR-11M types with good componentness and low inertia of the working part should preferably be used on mountain rivers and on hydraulic structures. Their instrument error (when the current meter is operated from a rod),due to pulsation of flow velocity and direction of flow, might vary within ± 5 per cent at the velocities corresponding to the linear part of the rating curve. When operated on lowland rivers (without pulsation of velocity and direction) irrespective of operating techniques (from a rod or cable), the instrument error is no more than ± 1.5 per cent in the rectilinear part of the curve and no more than ± 10 per cent in the curvilinear part.

Current meters with large propeller diameters (high moment of inertia), such as GR-21 and GR-21M types, should preferably be used on lowland rivers since they are reliable, durable and have low initial velocity. Irrespective of the operation technique (from a rod or a cable), their instrument error is no more than 1.5 per cent in the rectilinear part of the rating curve, and no more than ± 10 per cent in the curvilinear part.

When GR-21 and GR-21M component current meters are operated from a rod in a turbulent stream the error due to the influence of pulsation of velocity and direction varies from − 6 per cent to + 20 per cent, and for non-component current meters of the same type the error reaches ± 20 per cent.

When vertical axis current meters are operated from a rod or cable under unfavourable conditions the error may reach +40 per cent.

In hydrometric practice the types of current meters should be chosen with respect to the hydrodynamic characteristics of the water body and the technique of current meter operation (i.e. rod or cable) affecting the value of instrument error.

Linear extrapolation of the rating curve can be made only up to the critical velocity (v_{cr}) for each type of current meter.

Bearing in mind that the error of determination of the initial velocity (v_0) may reach 20-25 per cent, velocities from $2V_0$ to V_{cr} should be taken as the operational limits of a current meter.

DISCUSSION

André: Les résultats exposés sont-ils théoriques ou l'aboutissement de mesures sur le terrain ou en laboratoire ?

Burtsev: These are the results of the experimental and field work and of the laboratory work as well.

Accuracy of current meter measurements

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Abstract. The probable accuracy of a streamflow measurement made by current meter has been assessed by evaluating the separate error components. Non-random error components are: (1) errors in the current meter rating; (2) the error of velocity observation at a point: (3) the error from measuring velocity only at selected points in a vertical; and (4) the error from assuming that the depth and velocity vary linearly between stations in the cross-section. Values for each of these error components are known, based on statistical analysis of special measurements made for this purpose on many different streams. Statistical combination of these errors indicates that the probable error of a streamflow measurement made by the standard procedures is about 2 per cent. The information presented can be used to determine the optimum observation procedure to attain a desired accuracy.

EXACTITUDE DU JAUGEAGE AU MOULINET

Résumé. La précision probable d'une mesure de débit faite au moyen d'un moulinet est estimée en évaluant les divers facteurs d'erreur. Les diverses erreurs non accidentelles sont : (a) l'erreur de tarage du moulinet ; (b) l'erreur de mesure de la vitesse en un point ; (c) l'erreur de mesure de la vitesse effectuée seulement en des points choisis sur la verticale ; (d) l'erreur due à la présomption que la profondeur et la vitesse varient d'une façon linéaire entre les stations de la section transversale. Les valeurs de chacune de ces erreurs sont connues à partir des analyses statistiques de mesures spéciales effectuées à cette fin dans de nombreuses rivières. L'étude statistique de ces erreurs montre que l'erreur probable de la mesure d'un débit par les procédés classiques est d'environ 2 %. La présente étude peut être utilisée pour déterminer la méthode d'observation optimale en vue d'obtenir une précision désirée.

PRECISIÓN DE LAS MEDICIONES DE LOS CAUDALES

Resumen. La precisión probable de una medición de caudal en un curso de agua realizada con un medidor de corriente, ha sido calculada evaluando los distintos componentes del error. Los componentes de error no aleatorios son (a) errores en la calibración del medidor de corriente, (b) el error de la observación de velocidad en un punto, (c) el error producido al medir la velocidad solamente en los puntos seleccionados de una vertical, y (d) el error de suponer que la profundidad y la velocidad varían linealmente entre los distintos puntos de la sección transversal. Se conocen los valores para cada uno de estos componentes del error total basándose en el análisis estadístico de las mediciones especiales realizadas con estos fines en distintos cursos de agua. La combinación estadística de estos errores indica que el error posible en una medición de caudal en un curso de agua realizada con procedimientos normales, viene a ser del 2 %. Se puede utilizar la información presentada para determinar el procedimiento óptimo de observación, para alcanzar la precisión deseada.

ТОЧНОСТЬ ИЗМЕРЕНИЙ ВЕРТУШКАМИ

Резюме. Вероятная точность измерений расхода с помощью гидрометрической вертушки была рассчитана на основе анализа отдельных компонентов погрешности. Неслучайными компонентами погрешности являются : (1) погрешности при тарировании вертушки, (2) погрешность при наблюдении скорости в точке, (3) погрешность при измерении скорости только в выбран-

ных точках на вертикали и (4) погрешность в результате принятия допущения, что глубина и скорость изменяются линейно между вертикалями в поперечном сечении. Знакения для каждого из этих компонентов погрешности выведены на основании статистического анализа специальных измерений, проводившихся для этих целей на многих реках. Статистическая комбинация из этих погрешностей указывает, что возможная суммарная погрешность измерений расхода, выполняемых с помощью стандартных методов, составляет около 2 %. Представленная информация может быть использована для определения оптимального метода наблюдений, чтобы достигнуть желаемой точности.

INTRODUCTION

Streamflow is commonly measured by the current-meter method. The accuracy of these measurements depends primarily on the performance of the current meter and the number of observations of depth and velocity that are taken in a given cross-section. Although the absolute accuracy of a single measurement cannot be determined, some insight into probable accuracy can be gained from a study of the individual components of the total error.

A discharge measurement is made by observing the depth and velocity at a number of stations across a section. At each station the velocity is measured at one or more points over a time period. A time period of approximately 45 seconds is used in the United States.

Errors in measurement of depth and width are small and random in nature and thus have little effect on the accuracy of current-meter measurements. The primary sources of error are due to the following assumptions that are made in computing the discharge:

1. The rating of the current meter is correct and is applicable to the conditions of the measurement.
2. The velocity observed at a point is a true time-averaged velocity.
3. The ratio between the velocity at selected points in the vertical, and the mean velocity in the vertical is known.
4. The velocity and the depth vary with distance between stations in the section in a linear fashion.

The error resulting from each of these assumptions is considered separately in the statistical analysis that follows. The object of the analysis is to define the standard deviation of the discharge errors (in per cent of mean) that would result from single discharge measurements at a number of stream sites. The errors of approximately two-thirds of the measurements would lie within plus or minus one standard deviation. The concepts and definitions used are given in Table 1.

Each value of the ratio, R, in Table 1 is the partial discharge error, in per cent of mean for the entire section, associated with a particular source of error. For example, R_i is the error ratio for the section due to the instrument. Each value of the error ratio, r, is a velocity error in percent of mean at a point or station in the section. For example, r_s is the error ratio resulting from the shape of the vertical velocity-curve at a single station. The standard deviation, S, of a ratio R is a measure of the distribution of a particular ratio for a number of stream sites. The standard deviation, S_T, of the total discharge error, R_T, may be obtained from

$$S_T^2 = S_{R_i}^2 + S_{R_t}^2 + S_{R_s}^2 + S_{R_N}^2$$

PERFORMANCE OF CURRENT METERS

A current meter is rated by towing the meter through still water at various constant velocities and counting the number of revolutions of the rotor in a measured interval of time. It is assumed that this rating is applicable when the meter is used to measure velocity of turbulent flow in stream channels. Errors may be introduced by this assumption or by inaccuracies in the still-water rating.

Repeatability of still-water rating data. A large number of runs were made with two Price cup-type meters at each of several different car velocities to establish the precision or repeatability of the data obtained in the towing tank. The variation in velocity indicated by the current-meter rotors is shown in Table 2. The standard deviation (as a percentage) decreases as the velocity increases and is less than 1 per cent at velocities greater than one foot per second.

Individual versus average ratings. The United States Geological Survey has discontinued its practice of rating each current meter used in discharge measurement. This decision was based on a comprehensive study reported by Smoot and Carter (1969). The conclusions of this study were (a) that a single rating for a group of current meters of one manufacturer fits all observations as well as individual, repeated ratings

Table 1. *Statistical concepts and definitions used in analysis of discharge measurements*

Source of error	Partial error (section ratios)	Partial error (point ratios)	Standard deviation of section ratios
Instrument	$R_i = \frac{Q - Q_i}{Q} 100$	–	S_{R_i}
Velocity pulsation	$R_t = \frac{Q - Q_t}{Q} 100$	$r_t = \frac{v - v_t}{v} 100$	$S_{R_t} = S_{r_t}/\sqrt{N_p}$
Vertical velocity curve	$R_s = \frac{Q - Q_s}{Q} 100$	$r_s = \frac{V_m - V_s}{V_m} 100$	$S_{R_s} = S_{r_s}/\sqrt{N}$
Number of stations	$R_N = \frac{Q - Q_N}{Q} 100$	–	S_{R_N}

1. See Appendix for definition of symbols.

Table 2. *Repeatability of rating data for type AA Price current meters*

Velocity		Number of runs	Standard deviation of velocity Indicated by meter, as a percentage	
Feet per second	Meter per second		Meter 27	Meter 28
0.25	0.08	48	1.76	2.77
0.50	0.15	24	1.69	1.74
0.75	0.23	24	1.10	0.89
1.10	0.34	20	0.89	0.73
1.50	0.46	20	0.64	0.59

Table 3. *Standard deviation S_{r_t} (in per cent of error ratio, r_t, based on observations at 23 stream sites*

Point $1 - y/y_0$	Standard deviation for the indicated time, in seconds							
	15	30	45	60	90	120	180	240
0.20	5.7	4.7	4.2	3.8	3.3	2.9	2.5	2.0
0.60	7.5	6.4	5.6	5.2	5.2	4.6	3.6	3.4
0.80	10.3	8.0	7.2	6.4	5.6	5.0	4.4	3.6
$\frac{0.2 + 0.8}{2}$	5.9	4.6	4.2	3.7	3.3	2.9	2.5	2.1

Table 4. *Means and standard deviations of velocity ratio, r_s, at various relative depths*

$1 - y/y_0$	Mean ratio	Standard deviation
0.10	1.111	-
0.20	1.118	15.1
0.30	1.109	-
0.40	1.106	-
0.50	1.053	-
0.60	1.007	11.2
0.70	0.930	-
0.80	0.837	22.6
$\frac{0.2 + 0.8}{2}$	1.000	4.3

Table 5. *Standard deviation of ratio, R_N, for various values of N*

Number of stations N	Standard deviation S_{R_N}
8–11	4.2
12–15	4.1
16–20	2.1
21–25	2.0
26–30	1.6
31–35	1.6
104	0

Table 6. *Comparison of discharges from turbine rating and from stage-discharge rating[a] (or discharge measurements at gauge [b]), Columbia River, Washington*

Site	Number of comparisons	Average differences (percentage)
Bonneville Dam	7[b]	1.2
The Dalles Dam	13[b]	2.0
Chief Joseph Dam	2[b]	2.4
Chief Joseph Dam	30[a]	0.9
Ice Harbor Dam	40[a]	0.9
McNary Dam	74[a]	0.9

fit the observations for the individual meter and (b) the standard error of the single rating for a group of meters is due to random errors in observation rather than differences between meters.

The Geological Survey now carefully controls the manufacture of current meters by using a standard pattern for the current-meter yoke, and by stamping and forming dies for the rotor. A standard rating is used for the meters and only a sample of meters from a new group are rated to ensure quality control. All meter parts are considered interchangeable without change in rating so that damaged or worn parts could be replaced by new ones.

With the adoption of these standards, an appreciable amount of money has been saved by greatly reducing the number of current meter ratings. This has been accomplished without any sacrifice in accuracy of current-meter measurements.

Effect of turbulence. The applicability of still-water ratings to natural, turbulent flows is difficult to determine. Laboratory studies may be of little value because of the difference in scale of turbulence in the laboratory and in stream channels. A Price cup-type meter will rotate in still water when moved up and down but this movement may bear little relation to turbulent velocities in a stream channel.

Several authors have stated that Price type meters over-register and Ott type meters under-register in turbulent flow. However, comparative discharge measurements using Ott and Price meters reported by Townsend and Blust (1960) and by Carter and Anderson (1963) show no significant difference in the measured discharge that can be attributed to type of meter.

At the present time there is no evidence that turbulence has a significant effect on the performance of current meters.

Over-all meter error. The error ratio, R_i, due to the current meter is assumed to have a mean of zero and a standard deviation of 1 per cent if a different meter is used for each measurement. These values appear reasonable to the writer after considering the available evidence.

TIME SAMPLING OF VELOCITY

The velocity at any point in a stream is continually fluctuating in time even at a constant discharge. The point velocities observed during a current-meter measurement deviate in a random manner about the true mean velocity.

The nature of the fluctuations in velocity was studied at gauging station sites on twenty-three different rivers. Average velocities for consecutive time periods of 15, 30, 45, 90, 120, 180, and 240 seconds were observed at points corresponding to 0.2, 0.4, 0.6, and 0.8 depth for a one-hour period. The measuring sections ranged in depth from 2.4 to 26.7 ft (0.73 to 8.14 m) and the velocities from 0.43 to 7.9 ft/sec (0.13 to 2.41 m/sec). A statistical analysis of the data shows that:

1. Little correlation exists between the values of r_t at a point for consecutive time periods. The average correlation coefficient was 0.17.
2. Little correlation exists between the values of r_t observed simultaneously at different points in the section.
3. No correlation exists between the standard deviation of r_t and the depths or the velocities of the streams.
4. The velocity fluctuations can be assumed to be randomly distributed in time and space in a given cross section.

The standard deviations S_{r_t}, of the error ratios, r_t, about a mean of zero, for observations at twenty-three sites, are given in Table 3 for the various time periods and depths of observation. The standard deviation of r_t is given for velocity observations at a point. The standard deviation of error ratio, R_t, depends on the standard deviation, S_{r_t}, and the number of observation points, N_p, as follows: $S_{r_t} = S_{r_t} / \sqrt{N_p}$.

The error in a discharge measurement due to velocity fluctuation is much smaller than the values given in Table 3 because the velocity is observed at twenty-five or more points. For example, if 45-second readings are taken at the 0.2 and 0.8 positions on twenty-five stations the standard deviation is 0.8 per cent.

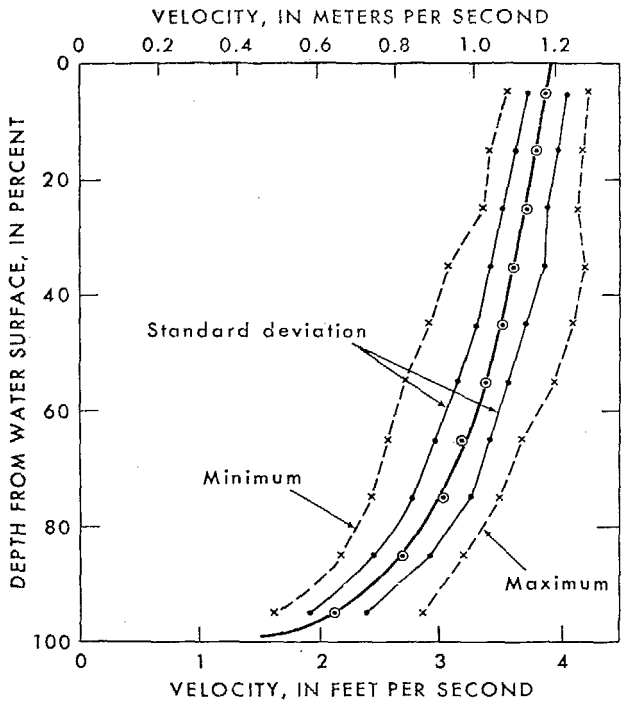


Figure 1. *The 66-minute vertical-velocity curve, vertical 280 in the measuring section of the gauging station, Columbia River below Priest Rapids Dam, 14 August 1963*

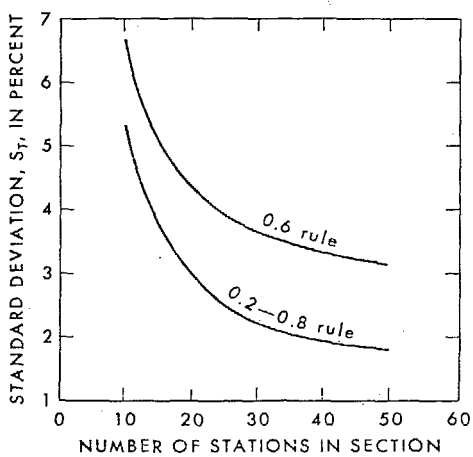


Figure 2. *Standard deviation of total error of discharge measurement*

POINT SAMPLING OF VELOCITY IN THE VERTICAL

In making a discharge measurement the velocity is observed at selected distances from the channel bed at a number of stations. In the United States it is common practice to observe the velocity at one or two points: at 0.6 depth, or at 0.2 and 0.8 depth from the water surface. The single-point method is used for depths less than 2.0 feet, and the two-point method for greater depths. The mean velocity in a vertical is then assumed to be equal to the velocity at 0.6 depth, or to the average of the velocities at 0.2 and 0.8 depth. This procedure is based on observations in natural channels, but is in close accordance with the logarithmic distribution function for a wide channel. Rouse (1950) shows that the use of the 0.2 and 0.8 rule or the 0.6 rule yields results less than 2 per cent higher than the logarithmic relationship, regardless of relative roughness or Reynolds number.

The shape of a vertical velocity curve actually varies from instant to instant because of turbulent fluctuations of velocity. In Figure 1 the central curve is defined by simultaneous observations at ten points in the vertical that were averaged over a period of 66 minutes. The standard deviation of one-minute observations and the maximum and the minimum one-minute observations at each point are also shown. Because there is little time correlation between velocities at any two points, it is evident that curves drawn through sets of one-minute observations could take a variety of forms. It has been shown, however, that regardless of the form of the curve, the 0.6-depth velocity or the average of the 0.2- and 0.8-depth velocities, closely approximates the mean in the vertical. Data obtained by the Geological Survey at over 100 stream sites are summarized in Table 4. The ratios of the average 0.3- and 0.8-depth velocities to the mean in the vertical have a mean value of 1.000 and a standard deviation of 4.3 per cent. Further analysis of the data indicates that the average correlation coefficient, ρ, between ratios for a given section is 0.04. The standard deviation of the section ratio R_s, may be obtained from the following equation:

$$S_{R_s} = S_{r_s} \left[1 + (N - 1)\,\rho\right]^{1/2} / \sqrt{N}$$

in which N is the number of stations taken in the section.

POINT SAMPLING OF DEPTH AND HORIZONTAL VELOCITY DISTRIBUTION

In computing a discharge measurement, the depth and the velocity are assumed to vary linearly with distance between stations in the section. The error in a discharge measurement due to this assumption and to random errors in depth measurement are both related to the number of stations, N, at which the depth and velocity are measured. The standard deviation of the R_N ratios was defined by analysis of special discharge measurements made at 127 different stream sites.

The discharge measurement at each site included velocity and depth observations at more than one hundred stations in the section. The discharge at each site was computed using the data for 1/2, 1/4, 1/5, 1/7, and 1/10 of the total number of stations. The ratios R_N of these discharges to the discharge computed using all stations, after adjustment for velocity fluctuation errors, were tabulated. For any given number of stations the mean of the R_N ratios was near 1.00. The standard

deviation, S_{R_N}, of the ratios for all sites is given in Table 5 for various numbers of stations; it varies from 4.2 per cent for 8 stations to the assumed value of 0 for 104 stations.

STANDARD DEVIATION OF TOTAL ERROR

The standard deviation of the total error of discharge measurements may now be determined from the standard deviation of the partial ratios by use of equation 1. For example, if velocity observations are taken at 0.2 and 0.8 depth for an average time period of 45 seconds at thirty stations in the cross section, then

$$S_T^2 = (1.0)^2 + (4.2/\sqrt{30})^2 + (4.3\,[1 + (30 - 1)\,0.04]^{1/2}/\sqrt{30})^2 + (1.6)^2;$$

$$S_T = 2.3 \text{ per cent.}$$

Thus, if single discharge measurements are made by this method at a number of sites, the error of two-thirds of the measurements would be less than 2.3 per cent.

The general relationship between the standard deviation of the total error and the number of stations taken in the cross-section is shown in Figure 2 for the 0.6-depth rule, and for the 0.2- and 0.8-depth rule, for a velocity observation of 45 seconds.

A COMPARISON OF TWO METHODS OF MEASUREMENT

The absolute accuracy of current meter measurements cannot be determined regardless of the method used. Comparison of measurements made by different methods, however, gives some insight to accuracy. Discharge values obtained at five sites on the Columbia River by two different methods are compared below.

To obtain accurate powerhouse discharge ratings at five multiple-turbine dams on the Columbia River, the Corps of Engineers, United States Army, made discharge measurements in the bulkhead slots at each dam. Detailed velocity traverses were taken by using a battery of cosine-component type meters mounted on a rack. These measurements were supplemented by simultaneous observations of gross head, power output, and spiral case piezometer differential pressure. The probable maximum error of the turbine ratings based on this data was computed to be ± 1.4 per cent.

The Geological Survey operates a gauging station downstream from each of the dams. These stations are rated by standard discharge measurements using the Price current meter. A comparison of simultaneous discharges computed from the turbine rating and from the stage-discharge rating (or discharge measurements) at the gauge is shown in Table 6. The close agreement of these discharges provides some verification of the probable error of discharge measurements as presented in Figure 2. On the average, about one per cent more discharge was measured at the gauging stations than at the dams.

CONCLUSION

The information given herein can be used to determine the optimum observation procedure for a given accuracy. All errors, except meter errors, could be virtually eliminated by observing the depth and velocity at a large number of points in the section. Usually, however, the added cost cannot be justified. An accuracy of 2 per

cent to 3 per cent is entirely adequate for most purposes. The standard deviation of the total error of a typical discharge measurement has been shown to be 2.3 per cent for a 30-station cross-section, with 45-second velocity observations at 0.2 and 0.8 depths. Numerous comparisons of measurements with discharges computed for well-rated dams show agreement within these limits.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Carter, R. W.; Anderson, I. E. 1963. Accuracy of current meter measurements. *American Society Civil Engineers Proc., Jour. Hydraulics Div.,* Julv 1963, p. 105-115.

Rouse, H. 1950. *Engineering Hydraulics.* New York, Wiley.

Smoot, G.F.; Carter, R. W., 1968. Are individual current meter ratings necessary? *American Society Civil Engineers, Proc., Jour. Hydraulics Div.,* March 1968.

Townsend, F.W.; Blust, F.A., 1960. A comparison of stream velocity meters: *American Society Civil Engineers,* Vol. 86, no. HY 4, April 1960.

APPENDIX: NOTATION

The following symbols have been adopted for use in this paper:

N = number of observation stations in a section;
N_p = number of observation points;
Q = true discharge;
Q_i = measured discharge with error due to current meter;
Q_N = measured discharge with error due to number of stations;
Q_s = measured discharge with error due to shape of vertical velocity curve;
Q_t = measured discharge with error due to velocity fluctuation;
R_i = section error ratio due to current meter;
R_N = section error ratio due to number of sections;
R_s = section error ratio due to shape of vertical velocity curve;
R_t = section error ratio due to velocity fluctuation;
R_T = total error ratio for discharge measurement;
r_s = error ratio due to shape of vertical velocity curve at a station;
r_t = error ratio due to velocity fluctuations at a point;
S = standard deviation of an error ratio (subscripts indicate the particular ratio);
V_m = true mean velocity in vertical;
V_s = computed mean velocity in vertical;
v = true time averaged velocity at a point;
v_t = measured velocity at a point with error due to fluctuation of velocity;
y = distance from channel bed;
y_0 = depth of flow; and
ρ = average correlation coefficient.

DISCUSSION

André: 1. Avez-vous fait des comparaisons entre mesures au moulinet et jaugeage par dilution ? En France, dans de bonnes conditions, les écarts ne dépassent pas 2 à 3 %.

2. Fait-on des corrections dues à l'angle des filets liquides avec la perpendiculaire à la section de jaugeage dans le cas des moulinets à axe vertical ?

Carter: 1. We have made numerous comparisons of the dilution method with the current meter method with a rate of accuracy of about 2 or 3 per cent.

2. We choose measuring sections with the flow normal to a line across the stream. In case of an angle, the observed losses are corrected after determination of the angle.

Struyk: The discharge measurement includes the observation of depth and velocity in a number of stations in the cross-section. At a certain moment you use a part of the total number of stations. My question concerns the criterium used for the selection of these stations and for rejecting the others.

Carter: Depth and velocity have been observed at hundreds of stations. After noting the effect at a number of stations, we started the selection, at first using half of the stations, then one-quarter, and finally one-eighth of the stations.

Burtsev: What type of current meter has been used for the comparison of water discharges obtained with a current meter attached to a rod and suspended from a cable?

Carter: Measurements have been made from cables by wading and also from supporting structures such as bridges, in such rivers as the Mississippi River, one of the largest rivers in the world, down to small streams.

Botma: It has been stated that the average correlation coefficient for the analysis of time sampling of velocity between the values at a point for consecutive time periods is only 0.17. The magnitude of this correlation coefficient depends strongly on the time period, so I should like to know which time period was used in this case.

Carter: Between 10 and 45 sec. were used here.

Lambie: Mr. Carter gives in his paper a figure of 2.3 per cent for the accuracy of the velocity component of the measurement, using this figure for comparing each individual gauging. When deducing the accuracy of a discharge measurement, as well as taking the error in the velocity component, it is necessary also to take into account the possible systematic or random error in measurements of width or depth, for assessing the overall accuracy of a gauging; these have to be combined with the velocity error. For his analysis, did Mr. Carter take into account the error in measurement of width and depth?

A second question: Mr. Carter mentions that a better result in the definition of velocity is obtained by using $(0.2 \pm 0.8)/2$ instead of 0.6 depth. Is this not wholly due to the first method?

Carter: We did not include the errors due to measurement of depth and width in this paper. We feel that these errors are so small that they are negligible.

Second question: I think that you are right that much of this difference in the 0.2 and 0.8 and 0.6 depth is due to the total number of observations.

Garg: A previous (U.S.S.R.) paper mentions that the error may reach 40 per cent with a vertical axis current meter. Has some work been done in this direction as

Mr. Carter's paper reveals that at present there is no evidence of turbulence having a significant effect on this performance of a current meter?

Carter: This subject covers an extremely broad field and needs to be restricted. Some work has been done to try to determine this effect on a Price meter and on the Ott meter. It only seems possible in a laboratory. We obtained only doubtful results. This question of turbulence cannot be answered at this time. But I should like to add that there is no evidence to indicate that turbulence has any significant effect on the performance of a Price meter.

Subramanya: It appears that the fluctuation of velocity vectors in a vertical plane will have more effect on the performance of a cup-type meter than on a propeller meter. The author's comment on this aspect would be appreciated.

Carter: From a theoretical viewpoint this is certainly a possibility. But I think that we cannot answer the question at this time.

Volker: To what extent have the measuring sites been selected with respect to their suitability for measuring discharges?

Has any correlation been found between the magnitude of the error and the quality of the measuring site (straight stream course, turbulence, depth variation, etc.)?

Glanz: Mistake or deviation, my question would be which one and against which value? Since the discharge is unknown, the mistake against the arithmetic means can be related to several measurements. Could you please give a precise definition about the mistake mentioned.

Carter: As I stated above, the absolute error of a current meter measurement cannot be determined. We have spoken about relative errors of different measuring procedures. The best way to make discharge measurements would be diametrically. This would be about the most absolute measures you could use. But on large rivers this is impossible. So we do not know what the absolute error is. We are really talking about relative errors between various procedures.

Grindley: When the cost of calibration of a current meter approaches the cost of a new meter, it may be advantageous to replace a meter instead of calibrating it. Nevertheless, individual calibration, provided that it is accompanied by good maintenance, must be better than group calibration.

A perfect current meter would be one which caused little disturbance to the parameter it was designed to measure. It would also respond rapidly to pulsations of flow.

A meter with a helical propeller having practically neutral buoyancy meets some of the requirements.

In recent years there has been a tendency for current meters manufactured in the United Kingdom to have propellers made of low-density material. Both the Plessey and the Braystoke meters have propellers made of plastic. Neither of these meters requires anything but the simplest maintenance. There are no ball bearings or oil-filled compartments. These meters retain their individual calibrations but in certain applications may be used with a group calibration since at 95 per cent

confidence the calibrations of the propellers are all within 3 per cent of the mean calibration.

Rodier: Sur les grands fleuves d'Afrique avec des appareils et des méthodes différents de ceux du "Geological Survey", nous arrivons à la même conclusion: précision de 2 %. Mais nous tenons à attirer l'attention sur le point suivant. L'erreur n° 4 de M. Carter tient à l'emploi de la méthode des 2 points par verticale et au dépouillement par ordinateur. Cela nous paraît justifié pour les moyens et grands cours d'eau. Pour les petits cours d'eau à écoulement irrégulier dans l'espace, ces méthodes conduisent à des erreurs atteignant parfois 10 ou 15 % (valeur de l'erreur n° 4). Pour éviteer des interprétations linéaires trop importantes, nous utilisons pour ces petits cours d'eau (moins de 20 à 40 m^3/s) les vieilles méthodes de dépouillement par isotaches ou par paraboles et nous nous interdisons l'emploi du dépouillement par ordinateur (sauf s'il s'agit de contrôler de grandes quantités de vieux jaugeages lorsqu'on veut vérifier si certains ne contiennent pas d'erreurs de 50 à 100 % par exemple).

Wiesner: 1. With increasing utilization of the water resources in the river areas of the German Democratic Republic, the significance of the knowledge of the error in flow rates is emphasized, especially with regard to safeguarding the supply to users. The error in discharge measurement, which is generally an individual measurement, is contained in the measuring result and cannot usually be recognized as such. Under the natural conditions prevailing during discharge measurement, it is not possible to determine its magnitude and the difference between measuring value and effective value. If repeated measurements in measuring series are available in which systematic errors are eliminated by measuring equipment and method, it will be possible to determine the error by statistical means.

In connexion with the complex investigations into the run-off process in the River Spree, a number of measurements were made with the current meter and the results permitted the establishment of statistics of error.

We checked the results of discharge measurements obtained in four river sections in the River Spree.

The measuring cross-sections were defined in equidistant spacing in river sections with lengths of 240 m, 1 200 m and 1 600 m. The measuring cross-sections in the River Spree were located down stream from the Spremberg Dam. This arrangement, and the discharge control during the measurement period, ensured the condition of no-discharge variations during measurement.

In determining the measurement of cross-sections in equidistant spacing in a stretch, independent of bends, we assumed that the continuity equation

$$Q = v \cdot A$$

where (Q = discharge ; v = average velocity ; and A = cross-section is applicable to all cross-sections in the area. This decision served the purpose of eliminating any systematic measuring errors which could be caused by profile peculiarities during measurement of velocity in a cross-section.

2. The statistics of error supply the following datá on the average value and the coefficient of variation of the various measuring methods:

	Measuring method (percentage)		
	6-point	2-point	1-point
Arithmetic mean $\bar{x}$	100	100	100
Coefficient of variation C_v	3.6	4.3	5.3
Coefficient of skewness C_s	0.24	0.68	1.41

Assuming that discharge has been properly determined with the six-point method, and the two-point method does not yield any deviation in one direction which would not be permissible for practical purposes.

If the sampling function is considered as an estimate of the distribution function of the basic total, the probability of the measuring error to be expected in a discharge measurement with the current meter and in the presence of the conditions mentioned above can be listed as follows:

Probability	Relative error (percentage)		
$P(90)$	6-point	2-point	1-point
10	± 0.5	± 0.6	+ 3 . . . + 1.6
20	± 1.0	± 1.1	+ 3.6 . . . − 1.0
50	± 2.4	± 2.9	+ 5.8 . . . − 1.4
80	± 4.6	± 5.5	+ 9.0 . . . − 4.4
90	± 5.9	± 7.0	+ 11 . . . − 6.4

Measurements in the individual measuring cross-section were made according to the six-point method and with strict adherence to the rules for measurement. That means that 10 vertical measuring lines were arranged across the cross-section width and velocities measured directly below the water surface in 0.8; 0.6; 0.4 and 0.2 depth as well as immediately over the bottom. Minimum measuring time was 40 seconds.

Assuming the condition that the change of run-off in the time unit is zero ($dQ/dt = 0$), the n repeated measurements of a measuring series can be considered a sample so that the effective flow can be determined as arithmetic mean of the flow rates from the individual measurements

$$\bar{Q} = \frac{1}{n} \sum_{1}^{n} Q_i.$$

The error of the individual measurement will then be the difference between the discharge of the individual measurement and the arithmetic mean of the measuring series

$$m_{Q_i} = \bar{Q} - Q_i$$

and, by writing the error of the individual measurement as a relative error,

$$v_{Q_i} = \frac{\bar{Q} - Q_i}{\bar{Q}},$$

summation of the samples to a large sample of $n = 51$ elements is made possible. The statistics of errors led to the following result: the relative errors are arranged on the probability paper around a straight line; the errors follow the Gaussian normal distribution and are random errors.

Calibration of current meters in a submerged jet

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Abstract. The feasibility of calibrating current meters in the submerged jet issuing from a cone-like nozzle was investigated. Preliminary tests have shown that this rapid method is of acceptable accuracy and that it could lead to savings in manpower, construction, and maintenance costs.

The uniformity of the velocity field in the jet has been verified for velocities between 0.25 and 8.0 feet per second (0.08 to 2.44 metres per second). The repeatability of the operation of regular Type AA and Pygmy Price meters has been statisfactorily demonstrated. The performance of each meter in the jet, compared to the conventional towing-tank rating of that meter, was within rating tolerances presently accepted.

The design of the test facility and desirable design criteria for a permanent structure are described, including dimensions of the forebay, nozzle, and tailbox. The flow recirculation system and flow measurement facilities by weight and by volume are discussed.

ETALONNAGE DES MOULINETS DANS UNE VEINE SUBMERGEE

Résumé. Des recherches ont été effectuées sur la possibilité d'étalonner des moulinets hydrométriques dans un filet d'eau s'écoulant d'un ajutage conique. Les essais préliminaires ont montré que cette méthode rapide est d'une précision acceptable et permet de réaliser une économie sur la main-d'œuvre et sur les prix de construction et d'entretien.

L'uniformité du champ de vitesse dans un filet d'eau a été vérifiée pour des vitesses allant de 0,25 à 8 pieds par seconde (de 0,08 à 2,44 m/sec.). Le bon fonctionnement répété des moulinets du type AA et du pygmée Price a été démontré de façon satisfaisante. La performance de chaque moulinet dans un filet d'eau, comparée à l'étalonnage de ce même moulinet par remorquage dans un bassin, était d'une tolérance d'étalonnage acceptable.

Les essais et les normes à respecter pour une installation permanente font l'objet d'une description. On donne les dimensions de l'avant-canal, de l'ajutage et de la boîte d'orientation. Le système de recirculation de l'écoulement et les moyens de mesurer l'écoulement, en poids et volumes, sont soumis à un examen critique.

CALIBRACIÓN DE MEDIDORES DE CORRIENTE EN UN CHORRO SUMERGIDO

Resumen. Se ha investigado la viabilidad de calibrar los medidores de corriente en un chorro sumergido que sale de una tobera en forma de cono. Los ensayos preliminares han demostrado que este método rápido es de precisión aceptable y que podría conducir a ahorros en mano de obra y en los costos de construcción y mantenimiento. La uniformidad del campo de velocidades en el chorro se han verificado para velocidades comprendidas entre 0,25 y 8,0 pies/seg. (0.08 a 2,44 m/seg.). La reiteración de la operación en los medidores de tipo AA normal y Pygmy Price ha dado resultados satisfactorios. La conducta de cada medidor en el chorro, comparada con la calibración convencional en tanque de aforo del mismo medidor, se encontraba dentro de las tolerancias de calibración actualmente aceptadas.

Se describe el diseño de la instalación de ensayo y los criterios deseables de diseño para una estructura permanente, incluyendo las dimensiones del remanso delantero, de la tobera y de la caja de la cola. Se estudia el sistema de recirculación de flujo y las instalaciones de medición de caudal por peso y por volumen.

ТАРИРОВАНИЕ ВЕРТУШЕК В ЗАТОПЛЕННОЙ СТРУЕ

Резюме. Исследовалась возможность тарирования вертушек в затопленной струе, вытекающей из конической насадки. Предварительные опыты показали, что этот быстрый способ дает приемлемую точность и что он может дать значительную экономию средств. Однородность скоростного поля в струе была проверена для скоростей между 0,25 и 8,0 футов/сек. (от 0,08 до 2,44 м/сек). Повторность действия вертушек типа АА и малой Прайса была удовлетворительной. Показания каждой вертушки в струе сравнивались с обычной тарировочной кривой для этой вертушки, полученной в тарировочном бассейне, причем расхождение находилось в обычно допускаемых пределах.

Описана конструкция опытного устройства и даны требуемые расчетные критерии для постоянного устройства, включая размеры подходного бассейна, насадки и низового ящика. Описана водооборотная система потока и оборудование для весовых и объемных измерений расхода.

INTRODUCTION

The United States Geological Survey calibrates current meters by towing them at a constant rate of speed through a tank of sensibly still water, once in each direction at velocities between 0.25 and 8.0 feet per second (fps; equivalent to 0.076 to 2.44 metres per second). The relation between the number of revolutions per second of the meter bucket wheel, and the corresponding velocity of the meter (or the rating car), defines the calibration of the meter. This method of rating is costly and time consuming. Furthermore, some of the spread in standard deviation of repeated calibrations of the same meter (see Smoot and Carter, 1968) can be attributed to uncertainties of the method, such as threads of circulatory currents stirred up in the water by previous passages; or persistent thermal currents in the tank, as discussed by Dickman (1951). In some installations, variations in the electric power or in the rails could affect the operation of the tow car.

One method of overcoming some of these objections would be to rate a current meter by placing it in the uniform velocity field of a submerged jet issuing from a contracting conical section. Water would be introduced into a head tank, schematically shown in Figure 1, and, after baffling, it would smoothly enter a conical section. The jet would be submerged in a tailbox, which would also be baffled to prevent undesirable currents. Getaway flow could be recirculated. Velocities could be easily changed and flow could be quickly stabilized. At each established velocity, many meters can be successively rated very quickly, with a consequent saving in costs.

In order to explore the feasibility of such a procedure, a contract was let with the Water Research Laboratory of Utah State University, Logan, Utah, United States, with specific information to be provided on the following:

(a) Method of introducing water into the head tank (free fall or under pressure;

(b) Size of head tank or stilling basin and the amount and type of baffling required to achieve smooth, nonturbulent entry into the contracting cone;

(c) Shape and size of contracting cone and orifice;

(d) Uniformity of distribution at the rating section;

(e) The optimum location of the meter with respect to the orifice and the mounting with the meter is supported;

(f) Effect of inserting the meter into the jet;

(g) Size required for the basin between the orifice and the weir and the amount and type of baffling required.

Table 1. *Comparison of jet core velocities as determined by pitot traverses and volumetric weighings, to centerline velocity as determined by independently calibrated Price current meters*

Core velocity	Type AA 272466	Type AA 272467	Type AA 272468	Type AA 261388	Type AA 261392	Pygmy 273437	Pygmy 273443	Pygmy 208791
0.294				0.291	0.292			0.306
0.557				0.559	0.560			0.562
0.900				0.870				
0.977				0.971	0.966			0.981
1.16				1.18	1.18			1.20
1.19	1.18	1.18	1.17			1.17	1.16	
1.25				1.25	1.24			
1.47				1.47	1.48			
1.54				1.55	1.53			1.55
1.59	1.59	1.60	1.60			1.60	1.59	
1.65				1.63				
1.95	2.01	2.02	2.02			2.02	2.03	
2.36				2.31	2.35			
2.35				2.36	2.35			2.36
2.78	2.77	2.76	2.78			2.81	2.76	
3.23								3.17
3.29				3.28				
3.81				3.71	3.87			
4.93				5.01				
5.27				5.20	5.35			
5.40				5.38	5.36			
6.16				6.15	6.16			
8.44				8.44	8.48			
8.55				8.26	8.50			
8.59				8.52	8.44			

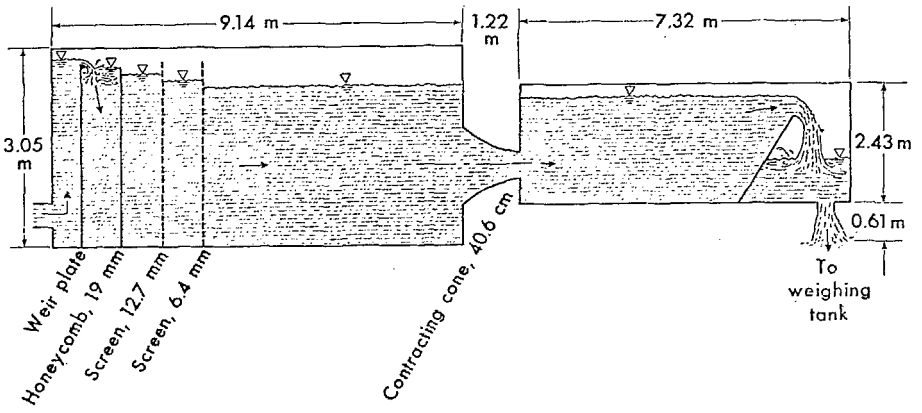


Figure 1. *Calibration system*

TEST FACILITIES

In the preliminary studies at Utah State University, a prototype-size arrangement was built with a forebay 2.44 m wide, 3.05 m high, and 9.14 m long (see Fig. 1). A tailwater tank was constructed 7.32 m long, 1.83 m high, and with a width varying from 1.22 m at the entrance to 0.91 m wide for the last 4.57 m of tank length. Between the two tanks, a converging cone was installed with an inlet diameter of 101.6 cm, an outlet diameter of 40.6 cm, and a length of 1.22 m.

This facility, designed largely on engineering judgment, was adequate for the tests. Much was learned in this prototype-size flume to adjudge the suitability of its design; it provides a good base for the design of a permanent rating structure.

Forebay. Flow, introduced into the bottom of the forebay from a 35.6-cm inlet pipe, rose 2.59 meters to spill freely over a weir plate; thus discharge would always remain constant over the weir and would be unaffected by any depth changes near the entrance to the cone. The weir helped spread the inflow laterally. To dissipate the turbulence of high discharges, a 5.1 cm long honeycomb with 19 mm hexagonal openings was placed 0.91 m downstream from the weir plate. Two screens, one being 12.7 mm wire cloth with 76 per cent open area and the other 6.4 mm wire cloth with 66 per cent open area, were placed 1.83 m and 2.74 m, respectively, downstream from the weir plate. The honeycomb and screens helped in the breakdown of eddies and in the reduction of surface waves for the remaining 5.8 m of the head tank.

In the 5.8 m between the last screen and the cone entrance, the flow cross-section was at a constant-flow depth, and a width, both of 2.44 m. It was desired to have a low level of turbulence in the submerged jet, with the approach velocity to the cone to be less than 7.62 cm/sec. For the maximum rating velocity of 2.44 m/sec, the approach velocity in this forebay is, therefore, 5.5 cm/sec. This general arrangement of the forebay proved to be satisfactory insofar as size and amount and type of baffling were concerned.

Tailwater Tank. The depth of flow in the tailwater tank is controlled by an overflow gate near the end. This depth serves both to submerge the jet from the cone, and also to control the depth in the forebay. Water flowing over the gate passes through a hole in the bottom of the tank, is next conveyed to weighing tanks, and then flows out to the river.

Cone Design. In the preliminary test for a jet with a uniform velocity field at velocities between 0.25 and 8.0 fps (0.076 and 2.44 m/sec), three contractions were studied:

(a) A contracting cone with an entrance diameter of 61.0 cm, an outlet diameter of 20.3 cm, and a length of 61.0 cm;

(b) A contracting cone with an entrance diameter of 101.6 cm, an outlet diameter of 40.6 cm, and a length of 86.5 cm;

(c) A converging nozzle with an entrance diameter of 40.6 cm, an outlet diameter of 20.3 cm, and a length of 70.4 cm; this nozzle was tested with the 40.6 cm cone described in item (b) placed to precede it, in order to ensure a nearly uniform velocity distribution at its entrance.

The cones described in (a) and (b) above were designed after Smith and Wang (1944), who used an electromagnetic analogy to develop a geometric form of contracting cones having uniform velocity fields at the throat.

Velocity traverses were taken of the submerged jets from each of these contractions with a pitot tube having an outside diameter of 3.2 mm. The traverses were taken both vertically and horizontally at planes perpendicular to the jet at locations 3.8 cm upstream from the end of the contraction, at the free jet, and at 7.6 cm downstream.

Typical traverses for several velocities between 1.0 and 8.0 fps (0.30 and 2.44 m/sec) are shown in Figure 2 for the contracting 40.6 cm diameter cone. Distributions upstream and downstream from this location were equally uniform, establishing the exit of the 40.6 cm diameter cone as a suitable place in which to rate current meters in a prototype flume.

METER TESTING PROCEDURE

The best and obvious check on this method of rating meters is to compare the performance of a meter in the submerged jet with its still-water rating in a towing tank. Several each of Type AA and Pygmy Price current meters were tested for this purpose at various times. The meters had all been freshly rated in a towing tank, and had had negligible or no use thereafter.

After each velocity was set and the flow in the system stabilized, a pitot velocity traverse of the jet was taken at the end of the cone section. Then the meter was clamped in mid-jet position on a rod locked to the floor of the tailwater tank and held rigidly at the top. The vertical axis of the bucketwheel was placed at the downstream edge of the cone so that half the cups were within the cone, and half outside. The Type AA regular meters were rated with tailpiece.

The displacement of water caused by the meters was noted on a manometer board by recording the head-tank and tailwater-tank water-surface elevations before and after inserting the meter into the water. The change in the head across the cone, caused by the presence of the Type AA Price meter was found to vary from about 0.3 mm at a velocity of about 0.3 m/sec to 18.3 mm at a velocity of 2.44 m/sec. For the Pygmy meter, the head difference at all velocities up to 0.91 m/sec was less than 0.3 mm. These differences in the head across the cone caused by the placement of the meter in the jet translate to a difference of 3 per cent in the factor $\sqrt{(2gh)}$ at 2.44 m/sec. Mean jet velocity was not, however, computed by this means, but, rather, by volumetric weighing measurements, and by pitot velocity traverses.

For each meter, the repeatability of performance was checked specifically, by timing the number of revolutions for periods of at least 40-60 seconds, over and over for up to one hour. Concurrently with this series, frequent volumetric weight measurements were made of the flow. The results of this series of tests were very useful in making changes in design of the water supply system.

WATER SUPPLY SYSTEM

Originally, water came directly from a reservoir adjacent to the laboratory through several hundred feet of pipes, varying in diameter from 121.9 cm down to 35.6 cm. During the repeated tests of the current meters at one-minute intervals, the time required for the meter to rotate a set number of revolutions was noted to vary from test to test. The magnitudes of these cyclic flow fluctuations appeared to vary with velocity both in amplitude and in period. The cause was suspected to be slug flow in

the pipelines between reservoir and laboratory. The plumbing was accordingly changed so that the reservoir water first went to a constanthead tank inside the laboratory, and thence flowed into the test flume. Although this helped dampen the fluctuations somewhat, it still did not completely overcome the problem if the test flume happened to be the only experiment in the building using water. Therefore, water was allowed to flow through other branches from the main pipeline for the remainder of the tests.

Figure 3 (a) shows results for a Type AA Price meter at a velocity of about 8.5 cm/sec. The variation of flow represents an over-all amplitude of about 8 per cent with a clearly recurring 20-minute period. The scatter of points about the average (dashed) line is much less in magnitude. Later on the same day, after allowing more water to flow elsewhere in the laboratory, another Type AA Price meter was tested (see Fig. 3 (b)). The range of over-all scatter for a typical Price meter is now on the order of 3.5 per cent, but the range about the faint vestiges of a 20-minute cyclic variation (dashed line) is on the order of ± 1 per cent.

This water-supply problem has been mentioned primarily to point out its importance in the design of a permanent prototype rating facility. Careful attention to routing of the inflow through constant-level head tanks, and, perhaps, the introduction of water into the test flume in free fall rather than under pressure would undoubtedly help.

RESULTS OF METER TESTS

Current meters were tested in groups of similar types at several different occasions. Two Type AA Price meters were tested together in one series of tests; three other Type AA Price meters comprised another group; two Pygmy Price meters were tested together as a group on one occasion; and a single Pygmy Price meter was tested alone in another series. The submerged-jet test results for all meters are shown in Table 1 for various core velocities. Meter velocities shown are those computed by using the towing-tank rating for each meter. Core velocities were determined by either or both of two methods: (a) from pitot-tube velocity traverses, or (b) from division of volumetrically weighed discharges by the cross-sectional area of the cone.

The use of a pitot tube to measure core velocities less than 1 fps (0.3 m/sec) is unreliable because of the difficulty of reading on manometer tubes any difference in head less than about 0.2 in (0.5 cm). On the other hand, the weighing system was not fully automated, nor was it sufficiently large to handle discharges which would give core velocities of 5 fps (1.52 m/sec) or greater. Therefore, low velocities were determined by weighing; high velocities were determined by pitot tube. There was very good agreement between the two methods for all other velocities.

Since Figure 2 shows that core velocities as determined by pitot traverse are quite uniform above 1 fps (0.3 m/sec), and since they are verified by volumetrically determined velocites, it is assumed that the velocity distributions in the jet at lower velocities are equally uniform and are equally in agreement with volumetrically determined velocities.

The date in Table 1 are plotted in Fig. 4, with core velocity shown on the vertical scale. On the horizontal scale, the average of computed velocities for all Type AA Price meters, and also the average of computed velocities for all Pygmy Price meters are shown. The data points align well the 45° line of perfect agreement.

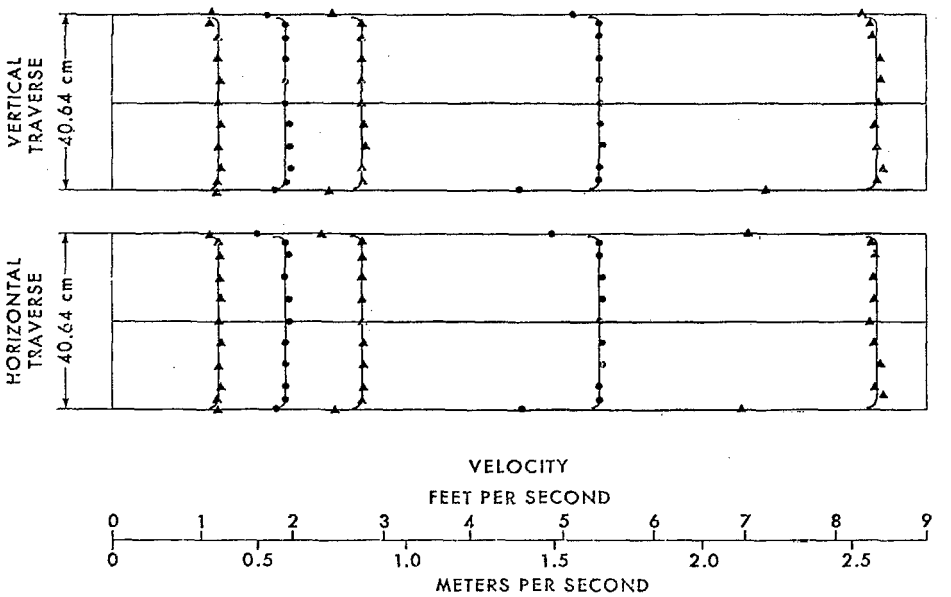


Figure 2. *Velocity distribution in submerged jet at throat of cone, for various velocities*

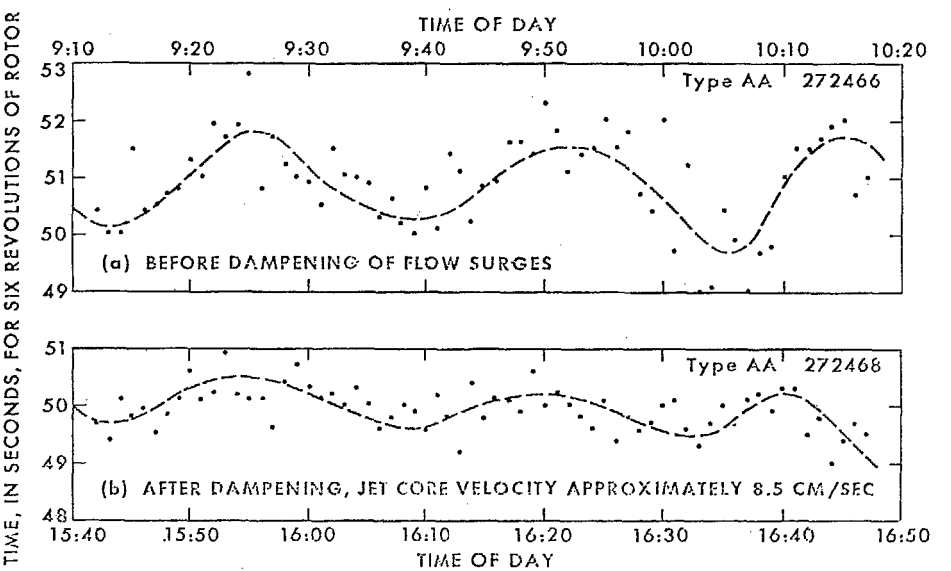


Figure 3. *Repetitive tests of type AA Price meters at one-minute intervals: (a) before dampening of flow surges, (b) after dampening*

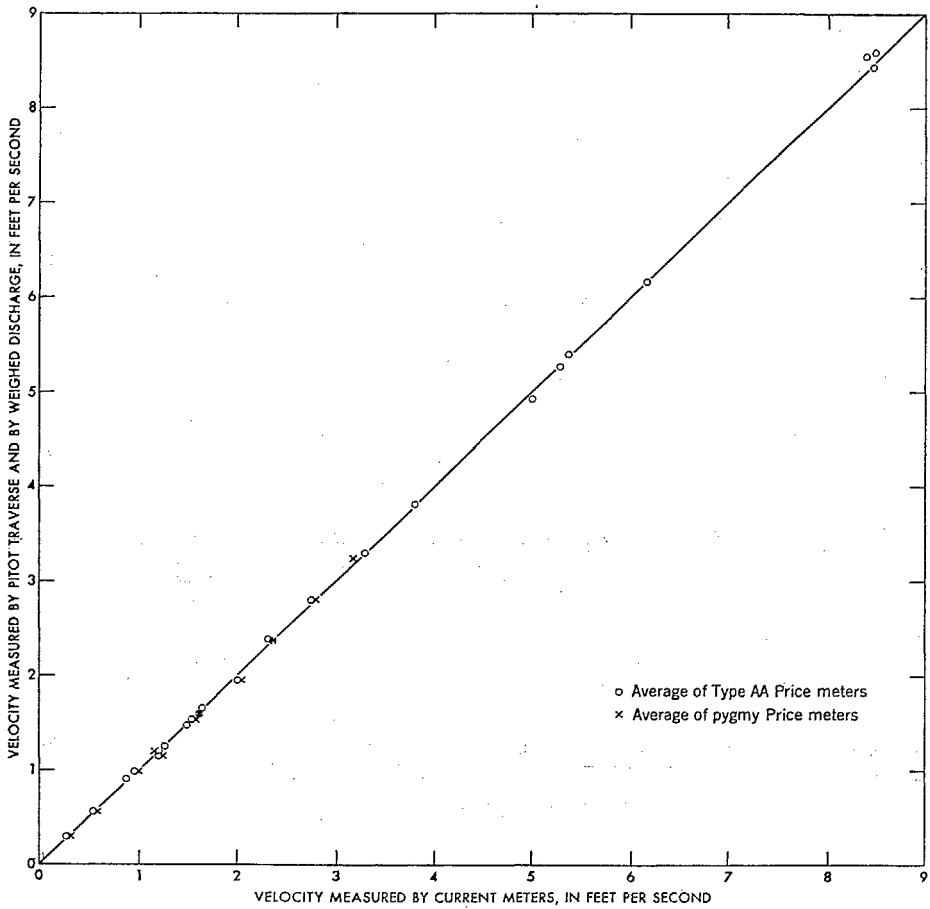


Figure 4. *Comparison of jet cone velocities determined by Price current meters, and by pitot traverses and weighed discharges*

CONCLUSIONS

The experience gained in these preliminary tests shows that the method is definitely practicable, overcoming difficulties associated with the towing-tank method of rating. Sufficient knowledge has been gained on each of the feasibility objectives to design the facilities and test procedures for such a permanent rating installation. It is envisioned that once a desired discharge is set, meters can be rated successively and rapidly for that velocity. Velocity would be determined by volumetric weighings and the velocity profiles would be checked occasionally by pitot traverses.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Dickmann, Robert H., 1951. The rating of water-current meters. *Water Power,* September. pp. 330-4, 341.

Smith, Richard, H.; Wang, Chi-Teh, 1944. Contracting cones giving uniform throat speeds. *J. Aeronautical Sciences,* vol. 11, no. 4, p. 356-360.

Smoot, George F., Carter, Rolland W., 1968. Are individual current-meter ratings necessary? *Am. Soc. Civil Engineers J.*, vol. 94, no. HY 2.

DISCUSSION

Gruber: 1. How did you obtain the distribution of velocity in the cross-section of the submerged jet?

2. How precise is the determination of its velocity?

Davidian: 1. One of the slides showed our velocity distribution both horizontally and vertically.

2. We had a problem of cyclic variations which were not related to the meters themselves. However, since sixty different tests were made within up to 60 minutes for each meter, we averaged them. The variation of an individual run about the curved test line through the entire series was of the order of about 1 per cent. The final results were of the order of about 1.5 to 2 per cent.

Oebius: The velocity distributions you showed were taken at about 2.5 and 7 cm from the orifice. At this distance, something has to be added to the submerged jet itself. The distribution varies from the values given by you until zero. The average distribution can be found only at a certain distance from the orifice which amounts to about 5.5 times the maximum diameter of the orifice.

Davidian: Upstream there would be a returning influence along the boundary. However, right at the mouth of the orifice the velocity distributions were uniform vertically and horizontally.

Oebius: The distribution of velocity you show was measured at a very small distance from the orifice. Farther away at about 5.5 times the diameter of the orifice, the velocity distribution which you show will no longer be present. The current meter should be placed at a small distance back from the orifice.

Davidian: Yes, the results therefore are shown exactly from just a few centimetres upstream and downstream from the emergence of the jet from the cone.

Campbell: In the described method of calibrating current-meters, you would not be able to determine the effect of various suspensions on the calibration of the meter due to the size of the orifice. Would you propose to do this separately or to suspend the meter always in such a manner that it would not be affected?

Davidian: This facility would have to hold the meter rigid. But we do not expect this to supplant the usual towing tank method.

The magnitude of errors at flow measurement stations

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Abstract. An assessment of the accuracy of hydrometric data produced from flow measurement stations is important to the users of the data. This is particularly so in the case of water resources development.

The paper uses a statistical approach and outlines simple statistical methods for obtaining the error in a single determination of discharge at both velocity-area stations and at weirs and flumes. In connexion with velocity-area stations, a method of obtaining the standard error of the stage-discharge curve is discussed and a statistical test for significance of check gaugings is demonstrated. Statistical definitions as they apply to hydrometry are included in an appendix.

GRANDEUR DES ERREURS DANS LES STATIONS DE JAUGEAGE DES DEBITS

Résumé. Il importe que les utilisateurs des données hydrométriques fournies par les stations hydrométriques (de jaugeage) puissent évaluer la précision de ces données. Cela est particulièrement vrai lorsqu'il s'agit de mise en valeur des ressources hydrauliques.

Dans cette communication, l'auteur aborde le problème sous un angle statistique et expose des méthodes statistiques simples qui permettent de déterminer l'erreur lors d'une seule mesure du débit, tant aux stations utilisant les moulinets qu'aux déversoirs et autres jaugeurs. A propos des stations utilisant les moulinets, l'auteur étudie une méthode permettant d'obtenir l'erreur type de la courbe de tarage et indique un test statistique grâce auquel il est possible de déterminer la signification des jaugeages de vérification. Des définitions statistiques s'appliquant à l'hydrométrie sont groupées dans une annexe.

MAGNITUD DE LOS ERRORES EN LAS ESTACIONES DE MEDICIÓN DE CAUDAL

Resumen. Es importante para los que utilizan los datos hidrométricos obtenidos en las estaciones de medición de caudal el conocer la precisión de dichos datos. Esto sucede especialmente en el caso del estudio de los recursos de agua.

Esta comunicación utiliza un enfoque estadístico y destaca los métodos estadísticos simples para averiguar el error en una única determinación de caudal, tanto en las estaciones de velocidad-área, como en los vertederos y otros dispositivos de medida. Respecto a las estaciones de velocidad-área, se discute un método para obtener el error normal en la curva de nivel-caudal y se muestra un ensayo estadístico para determinar la significación de las calibraciones de verificación. En el apéndice se han incluido las definiciones estadísticas utilizadas.

ВЕЛИЧИНЫ ОШИБОК НА СТАНЦИЯХ, ИЗМЕРЯЮЩИХ РАСХОДЫ

Резюме. Знание точности гидрометрической информации, полученной на станциях, измеряющих расходы воды, является важным для потребителей информации. Это особенно проявляется в случае развития водных ресурсов.

В статье используется статистический подход и описываются простые статистические методы для получения погрешности при единичном определении расхода как на станциях, измеряющих расход способом « скорость-площадь », так и на водосливах (лотках). Рассмотрен метод получения стандартной погрешности кривой зависимости расхода от уровня для станций,

измеряющих расход способом « скорость-площадь », и был продемонстрирован статистический прием установления важности поверочных измерений. Статистические определения, применяющиеся в гидрометрии, включены в приложение.

INTRODUCTION

By accuracy of flow measurement is meant the degree of agreement between the apparent flow as measured and the actual flow. The estimation of the accuracy of a measurement, therefore, consists in determining the likely magnitude of the difference between the apparent and the actual flow. It is not confined to a single measurement of the instantaneous flow but also to daily mean discharge, monthly mean discharge and average flow. The International Organization for Standardization (ISO) [refs. 1, 2] states.

"No measurement of a physical quantity can be free from uncertainties which may be associated with either systematic bias caused by errors in the standardizing equipment or a random scatter caused by a lack of sensitivity of the measuring equipment. The former is unaffected by repeated measurements and can only be reduced if more accurate equipment is used for the measurements. Repetition does, however, reduce the error caused by random scatter. The precision of the average 'n' repeated measurements is $\sqrt{n}$ times better than that of any of the points by themselves.

"When considering the possible uncertainty of any measurement of the discharge in an open channel, it is not possible to predict this uncertainty exactly but an analysis of the individual measurements which are required to obtain the discharge can be made and a statistical estimate made of the likely tolerance. A 95 per cent tolerance on measurement may be defined statistically as the bandwidth around the calculated value which, on an average of nineteen times out of twenty, can be expected to include the true value."

For computation of errors, therefore, the over-all tolerance should be based on the 95 per cent confidence level and this may be taken as approximately twice the standard deviation. However, care is required to ensure that individual or partial errors are used at the same confidence level. For current meter stations it is suggested that all errors are considered initially at 68 per cent confidence level and only at the final stages doubled to give the 95 per cent confidence level. In the case of weirs and flumes, however, where the coefficient of discharge is determined in the laboratory and random or systematic errors can be carefully controlled in the field, it is convenient to use partial errors at the 95 per cent confidence level.

KINDS OF ERRORS

Errors may be either random or systematic. The former are produced by purely chance fluctuations in conditions and the latter are associated with a particular instrument or method of measurement. In the discussion of errors in stream gauging it is assumed that the random errors involved are distributed according to the normal or Gaussian distribution. Fortunately any departure from this law has to be very severe before statistical tests based on it become invalid. In stream gauging certain systematic errors behave like random errors and may be assessed in a similar way. For

instance, in a measuring structure, small systematic errors may occur in the coefficient of discharge which may be attributable to variations in the surface finish of the structure or to the approach conditions, etc.

OVER-ALL TOLERANCE ON DISCHARGE

Estimating the tolerance on a single determination of discharge depends on estimation of the partial errors pertaining to various parts of the measurement. Some of these may be of widely varying ascertained errors and others may be the operator's own estimate of the accuracy of his work. Attention should be given, therefore, to evaluating partial errors as accurately as possible on site.

CONTRIBUTING (PARTIAL) ERRORS

For a single measurement of discharge by current meter the contributing errors are as follow [1] :

Errors (standard deviations expressed as percentages)
Errors in determination of velocity

X_m = Error due to the choice of the number of verticals
X_f = Error due to the choice of duration of exposure of current meter
X_0 = Error due to the choice of the number of points in a vertical.

Random errors

X'_b = Error in measuring width
X'_d = Error in measuring depth
X'_v = Error in measuring velocity

$$= \pm \sqrt{\left(\frac{Xf^2}{p} + X_{02}\right)}$$

where p is the number of points in the vertical at which velocities have been observed.

The overall percentage random error

$$X'_q = \pm \sqrt{\left[X'_{m2} + \frac{1}{m}(X'_{b2} + X'_{d2} + X'_{v2})\right]} \qquad (1)$$

where m is the number of verticals.

N.B. The above equation comes from the variance equation. If the individual observations are known then the variance of the mean is equal to the sum of the individual variances, i.e.

$$\sigma_T^2 = \sigma_1^1 + \sigma_2^2 + \ldots + \sigma_n^2 \qquad (1A)$$

where σ_1, σ_2, etc. are standard deviations of the mean $\sigma/\sqrt{N}$.

Systematic errors

X''_b = systematic error in instrument measuring width

X''_d = systematic error in instrument measuring depth
X''_v = systematic error in current meter.

The overall percentage systematic error

$$X''_q = \pm\sqrt{(X''^2_b + X''^2_d + X''^2_v)}. \quad (2)$$

The overall error is then

$$XQ = \sqrt{(X'^2_q + X''^2_q)}. \quad (3)$$

Random errors

Suggested values [1, 3] for the above errors are as follows. These values were suggested by the British Standard Committee after considering all the available data.

X_m Error due to the number of verticals

m	X_m
8	± 5 per cent
15	± 3 per cent
50	± 1 per cent

X_f Error due to choice of duration of exposure of current meter

For an exposure time of 40 seconds or 100 counts approximately the recommendation is $X'_f = \pm 6$ per cent.

X_0 Error due to the choice of the number of points in a vertical

Method	X_0
velocity distribution	0.5 per cent
2 point	3.0 per cent
1 point (0.6 depth)	3.5 per cent

X'_b Error in measuring width

Errors in measuring width, either between verticals or in total width, by line or winch are usually small and normally negligible and a value of ± 0.50 per cent is suggested.

X'_d Error in measuring depth

These are best determined on site but normally a tolerance of ± 1 per cent should be possible to attain at large values of depth and ± 3 per cent at shallow depths although repeated measurements may be necessary to reduce the tolerance, noting that the error for n repeated measurements is $\sqrt{n}$ times better than that for a single measurement. For example a measurement repeated 4 times halves the error of that of a single measurement.

Systematic errors

The possibility of systematic errors occurring in the instruments measuring velocity, depth, width, level, zero etc. should be carefully examined from time to time and

such errors corrected where necessary. The errors in X_b'' and X_d'' should be limited to ± 1/2 per cent but these errors are usually so small that they may be neglected. Again a field test is recommended to evaluate these errors.

X_v'' The error in the current meter

This error may be defined as the difference between the meter's tank rating and its performance in the field. There may also be a small systematic error in the tank calibration which is usually negligible. The rating of a current meter may take either the usual form of individual rating or the more recently proposed group rating. Lambie [4] and Smoot and Carter [5] suggested that a group rating is as accurate as an individual rating and that errors are primarily due to errors in the rating procedure. Part of the error may therefore be random and part systematic. Either way, however, makes little or no difference to the actual computation of the over-all error in discharge.

The tests carried out by Lambie [4] involved some 130 ratings totalling some 3,200 data points for 15 used Watts cup-type meters. Smoot and Carter [5] carried out tests with 140 new Price meters giving about 2,200 data points and Grindley in tests at the Hydraulics Research Station, England, used 18 new Watts cup-type meters and 120 new propeller meters from three different manufactures. From all of these tests the following table of percentage errors is suggested for both new cup-type meters and new propeller meters (standard errors at 68 per cent confidence level), for both individual and group ratings.

Velocity		Percentage standard error at 68 per cent confidence level (coefficient of variation)	
m/sec	ft/sec	Individual rating	Group rating
0.08	0.25	3	4
0.15	0.50	1	2
0.25	0.75	1	1
0.30	1.00	1	1
above 0.30	above 1.00	0.5	1

Example:

It is required to calculate the over-all tolerance XQ on an individual discharge measurement from the following particulars.

Number of verticals used	15 No.
exposure of meter	40 sec (2 exposures made)
method of measurement	2 point
average velocity	above 1 ft sec (0.30 m)
individual rating	

From the above information the partial errors (at 68 per cent confidence level) can be assessed.

Random:

$$X'_m = \pm 3\ \%$$

$$X'_f = \frac{6\%}{\sqrt{2}} \text{ (2 exposures)}$$

$$= \pm 4.25\ \%$$

$$X'_0 = \pm 3\ \%$$

$$X'_b = \pm 0.5\ \%$$

$$X'_d = \pm 2.5\ \%$$

$$X'_v = \pm \sqrt{\left(\frac{4.25^2}{2} + 3^2\right)}$$

$$= \pm \sqrt{18}\ \%.$$

Systematic:

$$X''_b = \pm 0.5\ \%$$

$$X''_d = \pm 0.5\ \%$$

$$X''_v = \pm 0.5\ \% \text{ (from table above).}$$

The overall random error in discharge is then

$$X'_q = \pm \sqrt{\left(X'^2_m + \frac{1}{m}\left(X'^2_b + X'^2_d + X'^2_v\right)\right)}$$

$$= \pm \sqrt{[3^2 + 1/15\ (0.5^2 + 2.5^2 + 18)]}$$

$$= \pm 3.25\ \%.$$

The overall systematic error in discharge is

$$X''_q = \pm \sqrt{(X''^2_b + X''^2_d + X''^2_v)}$$

$$= \pm \sqrt{(0.5^2 + 0.5^2 + 0.5^2)}$$

$$= \pm 1\ \% \text{ approx.}$$

The overall standard error is then

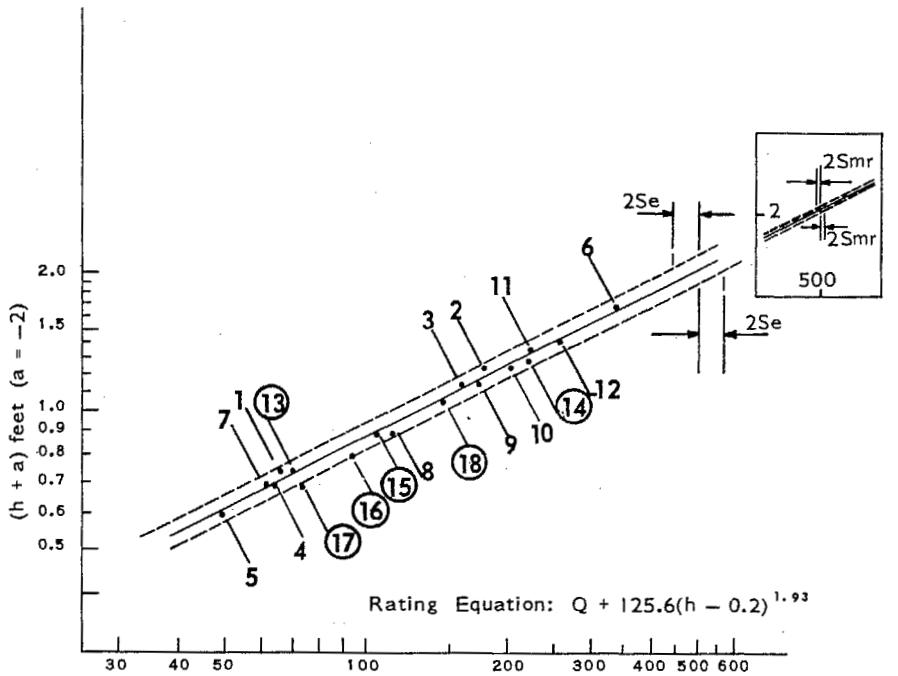
$$XQ = \pm \sqrt{(X'^2_q + X''^2_q)}$$

$$= \pm \sqrt{(3.25^2 + 1^2)}$$

$$= \pm 3.5\ \% \text{ approx.}$$

XQ at 95 % confidence level $= 2 \times 3.5$

$$= \pm 7\ \%.$$



NOTE:

95 per cent confidence limits for Se = 2 Se

$$= \frac{2\,Se \times Q}{100}$$

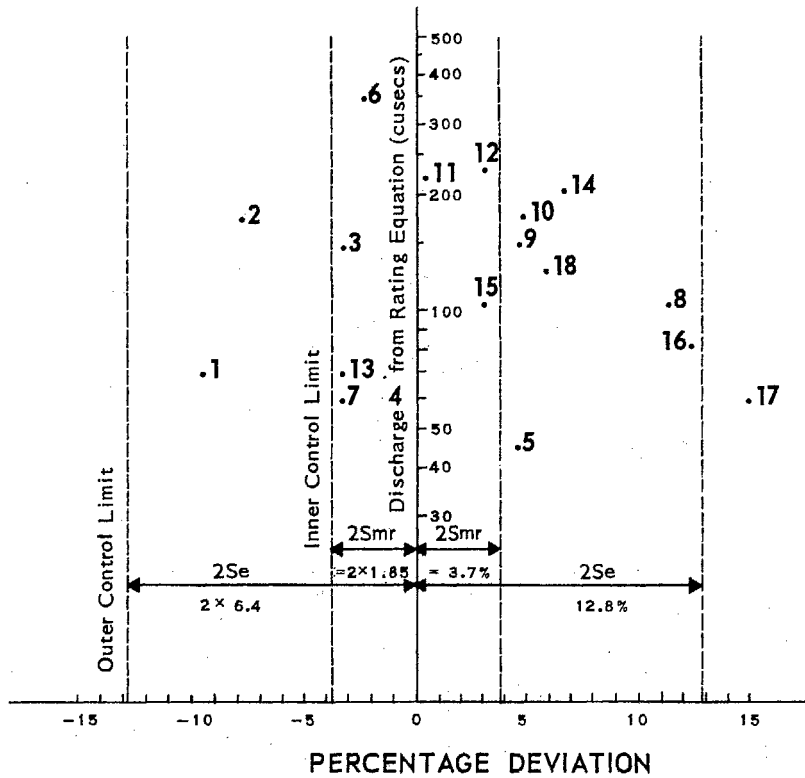
e.g. at Q = 500 cusecs

$$2\,Se = \frac{2 \times 6.4 \times 500}{100} = 64 \text{ cusecs}$$

Similarly for 2 Smr $= \frac{2 \times 1.85 \times 500}{100}$

$= 18.5$ cusecs

Figure 1. *Stage-discharge relation*



NOTE

2 Se represents the probable magnitude of error (at 95 per cent confidence level i.e. 19 times out of 20) in the individual gaugings made in determination of the rating equation.

2 Smr represents the probable magnitude of error (at 95 per cent confidence level i.e. 19 times out of 20) in the discharges taken from the rating equation.

Figure 2. *Deviation diagram*

That is on 68 times out of 100 (1 standard deviation) a single measurement of discharge made under these tolerances should be within ± 3.5 per cent of the true discharge; 95 times out of 100 (2 standard deviations) it should be within ± 7 per cent of the true discharge and 99.7 times out of 100 it should be within ± 10.5 per cent of the true discharge (3 standard deviations).

An approximate relationship between the over-all standard error and the number of verticals in the cross-section, after Carter [3] is shown graphically in Fig. 3 for the 0.6 method and the 0.2 and 0.8 method and one exposure of 40 seconds. The error in the single current meter is taken from the table. Although the over-all standard error can be reduced by repeating observations, as stated earlier, the curves in Fig. 3 show clearly that the error is also reduced by increasing the number of verticals. For the above example the 0.2-0.8 curve gives a standard error for 15 verticals of: 7.5 per cent at the 95 per cent confidence level.

STAGE-DISCHARGE CURVE

The stage-discharge curve being a curve of best fit should, unless some of the individual gaugings actually lie on the curve, be more accurate than any of the individual gaugings. One method of assessing this accuracy statistically is suggested as follows:

Fig. 1 shows a stages-discharge curve plotted on logarithmic graph paper from the gaugings numbers 1 to 12 in Table A (Appendix) [6]. Check gaugings 13 to 18 are also shown in Fig. 1 and indicated by circles. The equation of the curve is computed in the usual manner by the root mean square method [1] and is $Q = 125.6(h\text{-}0.2)$. The standard error of estimate (Se) expressed as a percentage (also referred to as the coefficient of variation) is calculated as shown in Tables A and B, to be 6.4 per cent at 68 per cent confidence level and the standard error of the mean relationship (Smr) is calculated to be 1.85 per cent at the same confidence level. This is more clearly presented in Fig. 2 where the central ordinate represents the stage discharge relation and the abscissa represents percentage deviations of the observed discharges from the central ordinate. The outer limits in Figs. 1 and 2 represent the tolerance on a single gauging at the 95 per cent confidence level and should include 19 out of 20 of the observations made in the determination of the stage discharge relation. Also, provided that there is no change in the hydraulic characteristics of the station, such as change in the control section, then 19 out of 20 of all check gaugings should fall between these limits. The inner limits on Figs. 1 and 2 represent the tolerance at 95 per cent confidence level on discharges taken from the stage discharge relation. That is in 19 cases out of 20 the true value of Q in the population lies between the limits of ± 2Smr. Usually 2Se or 2Smr may be conveniently computed for more than one range but normally more gaugings are taken at the lower end of a stage discharge curve and since 2Smr gets smaller as the number of gaugings increases its value may be more or less uniform for the entire range of stage.

The above computation may of course be conveniently performed by computer and Table D shows a copy of an actual computer output. Column 1 shows the level, or stage, in metres, column 2 the corresponding gauged flow in cubic metres per second, column 3 the corresponding discharge taken from the stage-discharge relation (computed from columns 1 and 2), column 4 the deviation of the gauged flow from the flow given by the stage-discharge relation (col. 3-col. 2) and column 5 gives the deviation expressed as a percentage of column 3.

The summary shows the logarithmic equation of the line of best fit as well as the standard error of estimate E at 95 per cent confidence limits. Note that log $C = 3.856$

is a natural logarithm to the base "e" and C is therefore equal to the anti-natural log of 3.856 which is 47.25. The equation is therefore:

$$Q = 47.25\,(H + 0.13)$$

with a standard error of estimate of ± 5 per cent at the 95 per cent confidence level. The standard error of the mean is not included in this particular programme but is $S_e/\sqrt{N} = 5/\sqrt{55} = 0.7$ per cent. We are 95 per cent confident therefore that the estimated stage-discharge relationship in this case is within ± 1 per cent of the true statistical relationship.

It is suggested that the above may afford not only a means of classifying stage discharge curves but also a means of assessing their statistical accuracy in terms of 2Se or 2Smr.

STUDENT'S t TEST FOR CHECK GAUGINGS

So long as check gaugings plot within the 2Se tolerance band without significant bias the established stage discharge relation may be considered valid. This means that, in each range, out of say ten check observations approximately five should plot on either side of the central ordinate in Fig. 2. Check gaugings should relate to a homogeneous period of time, i.e. a water year, or part of a water year, between changes in control. In most cases, however, check gaugings will amount to perhaps only a few per water year and in such circumstances a majority of the points may plot to one side of the central ordinate without necessarily indicating bias since there is no reason to suppose that additional data, if they had been available, might not have restored the balance. A suitable statistical test is the Student's t statistic which indicates whether or not the two sets of data may be considered to derive from the same statistical population. Student's t statistic is based on the property of sample means being distributed about the population mean with standard deviation $\sigma/\sqrt{n}$. If a sample is taken at random therefore, and it is found that it lies further from the population mean than $1.96\,s/\sqrt{n}$ (for 95 per cent confidence level), then it can be concluded that the sample is not likely to belong to the same population. The ratio of the deviation of the mean $\bar{X}$ of a particular sample of n items from an expected value E to its standard error of the mean $s/\sqrt{n}$, is known as Student's t:

$$\text{i.e.} \qquad t = \frac{\bar{X} - E}{s/\sqrt{n}}\,.$$

In the case under consideration it is required to test whether the two sets of data can be regarded as drawn from the one population that is, the difference in their means should not be significantly greater than zero. The above expression therefore becomes:

$$t = \frac{\bar{X}_1 - \bar{X}_2}{s/\sqrt{n}}$$

where $\bar{X}_1$ and $\bar{X}_2$ are the means of the two samples.

It can be seen from the above expression that the larger the discrepancy between the means the greater the value of t. Also, t has its own probability distribution, i.e. any specified values of t being exceeded with a calculated probability. Tables are

available showing the value t may reach for any given probability level and Table C is such a table abridged from *Statistical Tables for Biological Agricultural and Medical Research* by R.A. Fisher and F. Yates. In view of the nature of the data, it is considered that the probability level can be set tentatively at 10 per cent (0. 10), that is, there is a 90 per cent chance of being right, equivalent to a 10 per cent chance of being wrong. The odds therefore against values of t as big as or bigger than these occurring by chance are 9 : 1. The computed value of t is therefore compared to the value in Table C using the appropriate number of degrees of freedom, $n = N + N_1 - 2$. If t as computed is equal to or less than the tabulated value it may be accepted that both sets of data belong to the same population. A suggested tabulation and computation for t is given in Table B in which two examples are given. In the first example the check gaugings 13, 14, and 15 give a t value of 0.47 compared to a tabulated value in Table C of 1.77 for 13 degrees of freedom exceeded once in ten trials. The calculated value of t being less than the tabulated value, it can be concluded that the difference between the means is not significant. However the calculated t value for the 1961 Water Year check gaugings is larger than the tabulated t (2.79>1.77) and consequently the difference between the means of the two sets of data is significant. The action required in such a situation is either to carry out further check gaugings–which may improve the position–or, failing that, a possible re-calibration may be called for.

ERROR IN RECORDING STAGE

It is accepted that, when an individual gauging is made to establish the rating equation $Q\text{-}Ch\beta$ the stage h is measured with precision. However the stage actually recorded from day-to-day operation of the station will suffer from a random error. This error will vary with different types of recorder but for the punched tape recorder it may be taken as ±3 mm. To obtain the magnitude of the over-all error of an individual discharge this error in stage measurement–the largest of all the partial errors–must be included.

The tolerance on the gauged head h is the square root of the squares of the separate errors involved, that is, the accuracy of the instrument measuring head and the accuracy of the zero setting which may be taken as ±1 mm.

$$X_h = \frac{100}{h}\sqrt{(E_g^2 + E_z^2)} \tag{4}$$

where E_g is the error in head measurement and E_z is the error in the zero setting.

The over-all error in discharge is then

$$X_T = \sqrt{(2\mathrm{Smr}^2 + \beta^2 X h^2)} \tag{5}$$

at 95 per cent confidence level.

Example

To find the over-all accuracy of an individual discharge using the stage discharge curve in Fig. 1.

Let $(h + a)$ = 200 mm (0.65 ft)
where "a" is the stage correction factor, if any, corresponding to zero flow.

β = 1.93 (from Fig. 1)

2Smr = ± 3.7 %

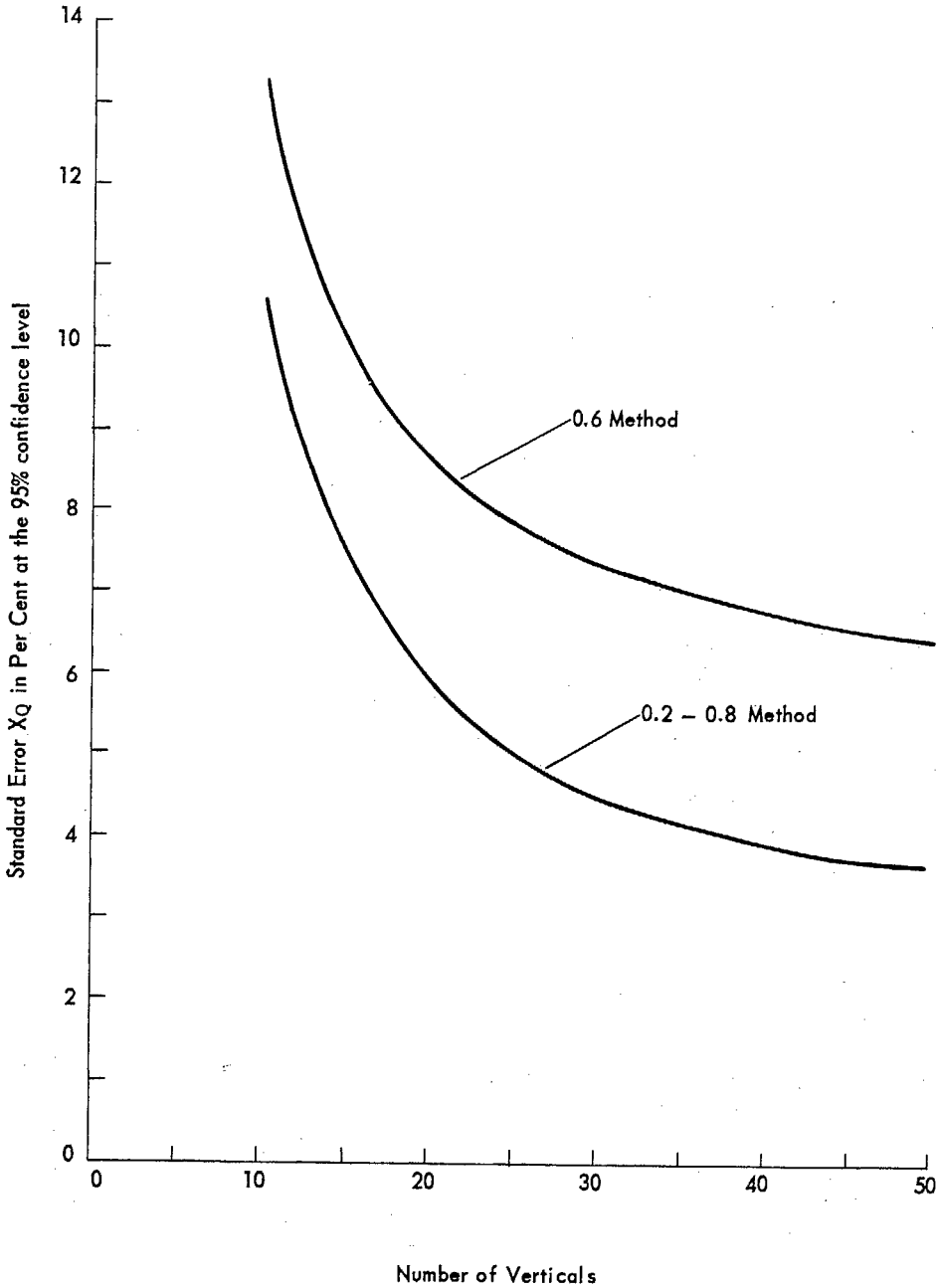


Figure 3. *Standard error of individual measurement of discharge performed by current meter*

$$= \pm 4\ \%$$

$$X_h = 100\ \frac{\sqrt{(3^2 + 1^2)}}{200}$$

$$= \pm 1.5\ \%$$

therefore, combined error $X_T = \sqrt{(4^2 + 1.93^2 \times 1.5^2)}$

$$= \pm 5\ \%.$$

Similarly it can be shown that at $(h + a) = 1$ m the over-all error for the individual discharge tends to a value of about ± 4 % or 2Smr.

In the case in point, therefore, we can say that at the 95 per cent confidence level individual discharges may be expected to fall within a tolerance band of ± 4 per cent of the true discharge at $(h + a)$ values above 1 m. At $(h + a)$ values below 1 m, the error increases until it is about ± 10 per cent at $(h + a) = 60$ mm.

WEIRS AND FLUMES

The tolerance X_T on the discharge may be calculated from the general error equation for weirs and flumes [7]

$$X_T = \pm \sqrt{(X_c^2 + X_b^2 + \beta^2 X_h^2)} \tag{6}$$

where X_c = percentage error in C_d, the coefficient of discharge.
X_b = percentage error in length of crest or width of throat.
X_h = percentage error in head.
β = index in the working equation (1.5 for weirs and flumes, 2.5 for V notches).

All tolerances in this case are taken at the 95 per cent level.

Percentage error in head X_h

$$X_h = \frac{100}{h} \sqrt{(E_g^2 + E_z^2)}$$

Percentage error in length of crest X_b

$$X_b = 100\ \frac{E_b}{b} \tag{7}$$

where E_b is the error in measuring length.

Calculation of error

Example: $h = 100$ mm (0.33 ft)

$X_c = \pm 2\ \%$

$\beta = 1.5$ for weir.

If the head is measured by punched-tape recorder the error in the head measurement may be taken as before as ± 3 mm. The zero of the gauge should be set to about ± 1 mm.

therefore, $$X_h = 100\ \frac{\sqrt{(3^2 + 1^2)}}{100}$$

$$= \pm 3\ \%$$

The length of crest can reasonably be measured to about 3 mm in 3 m

therefore,
$$X_b = \frac{100 \times 3}{3\,000}$$
$$= \pm 0.10\ \%$$

therefore, combined error in discharge

$$X_T = \pm \sqrt{(X_c^2 + X_b^2 + B^2 X_h^2)}$$
$$= \pm \sqrt{(2^2 + 0.10^2 + 1.5^2 \times 3^2)}$$
$$= \pm 5\ \%$$

Note: Recommended values for percentage errors in coefficients of discharge for weirs and flumes may be found in reference 7.

PRECISION OF THE DAILY MEAN DISCHARGE

The over-all or combined error considered so far has been in connexion with a single determination of discharge for any given stage. The daily mean discharge however is computed from several discharges over a period of 24 hours and its precision should be better than that for a single discharge. At a station equipped with a punched tape recorder the daily mean discharge is normally calculated from the arithmetic average of 96 fifteen-minute values of discharge. Now if these values of discharge can be treated as independent variables whose relative errors X_i, at 95 per cent confidence limits are known, then

$$\bar{Q} = \frac{\sum^{n} Q_i}{n} \tag{8}$$

and

$$[\bar{Q} X_{\bar{Q}}]^2 = \frac{\sum^{n} (Q_i X_i)^2}{n^2} \tag{9}$$

where $\bar{Q}$ is the average of n values of discharge Q_i, X_i is the error in Q_i, and X_Q is the error in the daily mean discharge.

Therefore,

$$X_{\bar{Q}}^2 = \frac{\sum^{n} [Q_i/\bar{Q}\, X_i]^2}{n^2}. \tag{10}$$

Note, equation 10 may be simplified to the following approximation:

$$X_{\bar{Q}}^2 = \frac{\Sigma (X_i)^2}{n^2} \tag{11}$$

and if all the values Q_i are equal, i.e. no fluctuation in stage, then

$$X_{\bar{Q}} = \frac{X_i}{\sqrt{n}}. \tag{12}$$

REFERENCES / REFERENCES

1. International Organization for Standardization (ISO). R. 748. 1969. *Velocity Area Methods.* Geneva.
2. International Organization for Standardization (ISO). R. 1100. 1969. *Establishment and Operation of a Gauging Station and Determination of the Stage Discharge Relation.* Geneva.
3. Carter, R. W.; Anderson, I. E. 1963. Accuracy of current meter measurements. *Proc. A.S.C.E.*, vol. 89, no. HY4.
4. Lambie, J. C. 1966. *The Rating and Behaviour of a Group of Current Meters.* I.C.E. Scottish Hydrological Group.
5. Smoot, G. F.; and Carter, R. W. 1968. Are individual current meter ratings necessary? *Proc. A.S.C.E.*, vol. 94, paper 5848.
6. Herschy, R. W. 1969. *The Evaluation of Errors at Flow Measurement Stations.* Water Resources Board. Technical Note No. 11.
7. British Standard 3680 Part 4A, 1964 and Part 4B, 1969. Part 4A: *Thin Plate Weirs and Flumes.* Part 4B: *Long Based (Broad-crested) Weirs.*

BIBLIOGRAPHY / BIBLIOGRAPHIE

Young, H. D. 1962. *Statistical Treatment of Experimental Data.* New York, Mc Graw-Hill Book Company Inc.

Hool, Paul G. 1963. *Elementary Statistics.* New York, John Wiley and Sons.

Simon, L. E. 1941. *An Engineer's Manual of Statistical Methods.* New York, John Wiley and Sons.

Mills, F. C. 1940. *Statistical Methods.* New York, Henry Holt and Company.

Chambers, E. G. 1958. *Statistical Calculation.* Cambridge University Press. British Standard 2846. 1957. *The Reduction and Presentation of Experimental Results.*

Langley, R. 1969. *Practical Statistics.* London. Pan Piper.

Moroney, M. J. 1963. *Facts from Figures.* Harmondsworth, Penguin Books Ltd.

Yeomans, K. A. 1968. *Applied Statistics,* Volumes 1 and 2. Harmondsworth. Penguin Books Ltd.

Topping J. 1968. *Errors of Observation and their Treatment.* (Institute of Physics), London. Chapman and Hall Ltd.

International Organization for Standardization. Recommendation No. 1438. *Liquide Flow Measurement in open Channels Using Thin Plate Weirs and Venturi Flumes.* (Draft.).

International Organization for Standardization. Recommendation R. 772. 1968. *Vocabulary of Terms and Symbols.* Geneva.

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APPENDIX: DEFINITIONS OF STATISTICAL TERMS

Normal distribution

Known also as the error law, the Gaussian law, the probability curve. In such a distribution the dispersion of the individuals, or items, about the average is measured in standard deviations and 68 per cent (actually 68.27) of the individuals lie within one standard deviation of the mean, 95 per cent (actually 95.45) lie within two standard deviations, and almost all (actually 99.73 per cent) lie within three standard deviations of the mean.

Standard deviation (s)

The standard deviation is a measure of the scatter of the items about the arithmetic mean of the sample. It is sometimes referred to as the root mean square of the deviations:

$$s = \sqrt{\frac{\Sigma(x)^2}{N-1}}$$

where x = the deviation of the item from the arithmetic mean ($\bar{X}$)
or $x = X - \bar{X}$
N = number of items.

Standard error of estimate (Se)

Also known as the "standard error", it is a measure of the variation or "scatter" of the points about the line of average relationship. It is similar to standard deviation except that the curve of relationship takes the place of the arithmetic mean.

$$\text{Se} = \sqrt{\frac{\Sigma(d)^2}{N-2}}$$

where d = deviation of the actual values from the computed values taken from the curve.

In the case of a stage discharge curve the "d" values are the difference between the discharge, as given from the curve, and the corresponding gauged discharge at the same stage. Dispersion can be measured in the same way as for standard deviation and if the distribution is normal, lines drawn at a vertical distance of one standard error of estimate above and below the line of regression and parallel to it will include about 68 per cent of the points. Parallel lines at two standard errors of estimate above and below the line of regression will include 95 per cent of the points, and lines drawn at three standard errors of estimate will include 99.73 per cent of the points.

Standard error of the mean relationship (Smr)

Also known as the "standard error of the mean", it is a measure of the probable accuracy of the computed mean relationship

$$\text{Smr} = \frac{\text{Se}}{\sqrt{N}}$$

In the case of a stage discharge curve, Smr indicates the probable accuracy of discharges picked off the curve from corresponding stages (i.e. the rating table discharges).

The Smr tolerance band is therefore much narrower than the Se band the latter which refers to the individual items and the band within which the individual items may be expected to fall. In a stage discharge curve these individual items may be gaugings or check gaugings.

Variances

The variance is the square of the standard deviation. It is a property of variance that if the variances of the individual observations are known then the variance of the mean is equal to the sum of the individual variances,i.e.

$$\sigma_T^2 = \sigma_1^2 + \sigma_2^2 + \sigma_3^2 \ldots + \sigma_n^2$$

where σ_1 σ_2,etc. are standard deviations of the mean.

Also the variance of the sum or difference of two independant random variables is equal to the sum of the variances,i.e.

$$\text{variance} = \frac{s_1^2}{N_1} + \frac{s_2^2}{N_2}$$

and if the samples S_1 and S_2 are assumed to be derived from the same population then the standard error of the difference of the means is given by $S_1 = s\sqrt{\frac{1}{N_1} + \frac{1}{N_2}}$

or $$S_1 = s\sqrt{\frac{N_1 + N_2}{N_1 N_2}}$$ as used in Table B.

Coefficient of variation (C.V.)

The standard error of estimate expressed as a percentage of the mean (as in Tables A and B) may be referred to as the coefficient of variation. In the same way the standard deviation may be expressed as a percentage of the mean and the coefficient of variation would in this case be $V = \frac{s}{\bar{x}} \times 100$ where $\bar{x}$ is the arithmetic average or mean. The advantage of using the coefficient of variation is that it is independent of units and can therefore be useful in comparing dispersions (see Fig. 3).

Degrees of freedom

A concept in statistical analysis imposing constraints on the system. In computing standard deviation one degree of freedom is lost and N should be replaced by (N-1). For a straight line two degrees of freedom are lost and in computing standard errors N should be replaced by (N-2). If two straight lines are required to locate a relationship four degrees of freedom are lost and if a transition curve is required to connect the two lines a further two degrees of freedom are lost and N should then be replaced by (N-6).

Student's t statistic

The ratio of the deviation of the mean x of a particular sample of n items from an expected value E, to its standard error of the mean,i.e.

$$t = \frac{\bar{x} - E}{s/\sqrt{n}}.$$

In the case of two means the expression becomes

$$t = \frac{\bar{X}_1 - \bar{X}_2}{s/\sqrt{n}}$$

where $\bar{X}_1$ and $\bar{X}_2$ are the means of the two samples.

Note that in Table B the standard deviation of the combined observations is

$$s = \sqrt{\frac{\Sigma(x_1 - \bar{x}_1)^2 + \Sigma(x_2 - \bar{x}_2)^2}{N_1 + N_2 - 2}}$$

where $(x_1 - \bar{x}_1)$ and $(x_2 - \bar{x}_2) = (d_1 - \bar{d}_1)$

and $t = d/S$ where $\bar{d}$ is the average percentage difference between the check gaugings and the rating equation and $S_1 = s\sqrt{\frac{N + N_1}{N \times N_1}}$.

Tolerance

The tolerance on a measurement may be defined statistically as the band width around the calculated value, which, on an average of 19 times out of 20, can be expected to include the true value. The tolerance can be taken to equal approximately twice the standard deviation.

N.B. More precise definitions may be obtained from text books on mathematical statistics but it is suggested that the above definitions are suitable for application to hydrometry.

NOTATION FOR TABLES A AND B

d - The percentage deviation of a measured discharge used in the construction of the rating equation from the value given by the rating equation for the corresponding gauge height.
d_1 - The percentage deviation of a check measurement of discharge from the value given by the rating equation for the corresponding gauge height.
$\bar{d}_1$ - The arithmetic mean of the deviations (d_1) of a series of check measurements of discharge; also, assuming the rating equation represents the best mean of the original observations, the difference between that and the mean of the check observations.
N - The number of observations used in constructing the rating equation.
N_1 - The number of check observations.
Se - Standard error of estimate.
Smr - Standard error of the mean relationship.
s - The best estimate of Standard Deviation of the Population of all the discharge measurements.
S_1 - The standard error of the difference between the means of the two arrays.
t - A statistic to test the significance of the difference between the sample mean and its hypothetical value based on Student's distribution.

Table A – *Percentage variation of actual gaugings from discharge shown by rating equation*

Date	No.	Gauge height	Q_2 Gauging	Q_1 Rating equation	$Q_2 - Q_1$	$\frac{Q_2 - Q_1}{Q_1} \times 100 = d$ or d_1	$d_1 - \bar{d}_1$	d^2 or $(d_1 - \bar{d}_1)^2$
Discharge curve								
3.3.59	1	0.95	65	72.4	– 7.4	– 10.2		104
4.4.59	2	1.45	177	193	– 16	– 8.3		68.9
4.4.59	3	1.35	159	165	– 6	– 3.6		13
16.5.59	4	0.90	63	63.1	– 0.1	–		–
8.6.59	5	0.80	49	46.9	2.1	4.4		19.4
12.8.59	6	1.90	341	349	– 8	– 2.3		5.3
3.9.59	7	0.90	61	63.1	– 2.1	– 3.3		10.9
21.9.59	8	1.10	114	102	12	11.7		137
22.9.59	9	1.35	172	165	7	4.3		18.5
22.9.59	10	1.45	202	193	9	4.6		21.2
22.9.59	11	1.55	225	224	1	0.4		0.2
22.9.59	12	1.62	255	247	8	3.2		10.2
$N =$	12						$\Sigma d^2 =$	408.6
Check gaugings —1959-60								
4.12.59	13	0.95	70	72.4	– 2.4	– 3.3	– 5.1	26
12.2.60	14	1.50	220	208	12	5.8	4.0	16
21.3.60	15	1.10	105	102	3	2.9	1.1	1.2
$N_1 =$	3					$\Sigma d_1 =$ 5.4	$\Sigma(d_1 - \bar{d}_1)^2$	43.2
						$\bar{d}_1 =$ 1.8		
Check gaugings — 1960-61								
20.10.60	16	1.00	92	81.6	10.4	12.7	2.1	4.4
16.2.61	17	0.90	72	63.1	8.9	14.1	3.5	12.3
27.2.61	18	1.25	145	138	7.0	5.1	– 5.5	30.3
$N_1 =$	3					$\Sigma d_1 =$ 31.9	$\Sigma(d_1 - \bar{d}_1)^2 =$	47
						$\bar{d}_1 =$ 10.6		

Table B – *Standard error of estimate. Standard error of mean. Test of check gaugings for bias*

Se and Smr	1959-60 check gaugings
Range of h = 0.80 to 1.90	Test of check gaugings for bias
Σd^2 (from Table A) = 408.6	$N_1 = 3$, $\bar{d}_1 = 1.8$, $\Sigma(d_1 - \bar{d}_1)^2 = 43.2$
N = 12	$s = \sqrt{\frac{\Sigma(d^2) + \Sigma(d_1 - \bar{d}_1)^2}{N + N_1 - 2}}$ = 5.9
$N - 2$ = 10	$S_1 = s\sqrt{\frac{N + N_1}{N \times N_1}}$ = 3.8
$\Sigma \frac{d^2}{N-2}$ = 41	$t = \bar{d}_1/S_1 = 1.8/3.8$ = 0.47
$Se = \sqrt{\frac{d^2}{N-2}}$ = 6.4 %	For $(N + N_1 - 2)$ degrees of freedom
$Smr = \frac{Se}{\sqrt{N}}$ = 1.85 %	(i.e. 13) $t_{0.10} = 1.77$ > 0.47 (Table C)
	∴ Action required None
	1960-61 check gaugings
	$N_1 = 3$, $\bar{d}_1 = 10.6$, $\Sigma(d_1 - \bar{d}_1)^2 = 47.0$
	$s = \sqrt{\frac{\Sigma(d^2) + \Sigma(d_1 - \bar{d}_1)^2}{N + N_1 - 2}}$ = 5.9
	$S_1 = s\sqrt{\frac{N + N_1}{N \times N_1}}$ = 3.8
	$t = \bar{d}_1/S_1 = 10.6/3.8$ = 2.79
	For $(N + N_1 - 2)$ degrees of freedom
	(i.e. 13) $t_{0.10} = 1.77$ < 2.79
	∴ Action required further checks, poss. re-calib

Table C – *Values of* $t_{0.10}$

Deg. freedom	t	Deg. freedom	t	Deg. freedom	t
1	6.31	11	1.80	21	1.72
2	2.92	12	1.78	22	1.72
3	2.35	13	1.77	23	1.71
4	2.13	14	1.76	24	1.71
5	2.02	15	1.75	25	1.71
6	1.94	16	1.75	26	1.71
7	1.90	17	1.74	27	1.70
8	1.86	18	1.73	28	1.70
9	1.83	19	1.73	29	1.70
10	1.81	20	1.73	30	1.70

(Abridged from Table III of *Statistical Tables for Biological, Agricultural and Medical Research* by R.A. Fisher and F. Yates. Edinburgh, Oliver and Boyd.)

Table D

Stage (m)	Q_2 Gauged flow (m^3/s)	Q_1 Rating equation (m^3/s)	$Q_1 - Q_2$ Deviation (m^3/s)	$\frac{Q_1 - Q_2}{Q_1} \times 100$ (per cent)
3.765	320.000	328.979	8.979	3
3.284	277.000	272.472	– 4.528	– 2
3.296	276.000	273.842	– 2.158	– 1
3.254	273.000	269.055	– 3.945	– 1
3.234	272.000	266.784	– 5.216	– 2
3.247	271.000	268.259	– 2.741	– 1
3.310	270.000	275.444	5.444	2
3.221	270.000	265.311	– 4.689	– 2
3.260	268.000	269.737	1.737	1
3.243	267.000	267.805	0.805	0
2.827	223.000	221.856	– 1.144	– 1
2.778	220.000	216.617	– 3.383	– 2
2.322	171.000	169.723	– 1.277	– 1
2.170	160.000	154.874	– 5.126	– 3
2.095	150.000	147.700	– 2.300	– 2
2.000	141.000	138.760	– 2.240	– 2
2.012	133.000	139.880	6.880	5
1.906	130.000	130.082	0.082	0
1.938	129.000	133.017	4.017	3
1.888	127.000	128.439	1.439	1
1.908	127.000	130.264	3.264	3
1.844	127.000	124.450	– 2.550	– 2
1.869	126.000	126.712	0.712	1
1.870	120.000	126.803	6.803	5
1.820	119.000	122.290	3.290	3
1.638	107.000	106.289	– 0.711	– 1
1.535	95.900	97.537	1.637	2
1.526	94.600	96.783	2.183	2
1.376	88.300	84.481	– 3.819	– 5

continued overleaf

Table D

Stage (m)	Q_2 Gauged flow (m^3/s)	Q_1 Rating equation (m^3/s)	$Q_1 - Q_2$ Deviation (m^3/s)	$\frac{Q_1 - Q_2}{Q_1} \times 100$ (per cent)
1.376	87.200	84.481	− 2.719	− 3
1.350	81.100	82.400	− 1.300	2
1.154	67.400	67.229	− 0.171	0
1.166	66.900	68.130	1.230	2
1.137	65.400	65.957	0.557	1
1.006	58.800	56.408	− 2.392	− 4
0.968	55.600	53.723	− 1.877	− 3
0.930	53.000	51.078	− 1.922	− 4
0.916	51.400	50.113	− 1.287	− 3
0.893	47.300	48.541	1.241	3
0.808	42.900	42.863	− 0.037	0
0.781	42.700	41.104	− 1.596	− 4
0.803	41.100	42.536	1.436	3
0.764	39.300	40.009	0.709	2
0.731	38.400	37.907	− 0.493	− 1
0.709	37.300	36.525	− 0.775	− 2
0.706	36.600	36.337	− 0.263	− 1
0.710	36.000	36.587	0.587	2
0.676	32.400	34.481	2.081	6
0.549	26.600	26.955	0.355	1
0.483	23.600	23.271	− 0.329	− 1
0.437	20.700	20.802	0.102	0
0.425	20.500	20.172	− 0.328	− 2
0.408	19.100	19.289	0.189	1
0.385	18.200	18.113	− 0.087	0
0.383	17.900	18.012	0.112	1

Summary: The line that gives the best estimate of Q, for a given H, with approx 95 per cent confidence limits is given by

$$\log (Q) = \log (C) + \alpha \log (H + A) \pm E$$

between the limits 0.383 and 3.765 where $\log C - 3.8555$, $A = 0.13$, $\alpha = 1.428$ $E = \pm 5$ and $C = 47.25$ and $Q = 47.25\ (H + 0.13)^{1.428}$.

DISCUSSION

Smoot: I have to comment on the table concerning the errors. In the United States we have run some extensive tests on the calibration of current meters by groups or individually. Our experience would indicate that our group ratings are equally as accurate as individual ratings and we have approximately the same standard for either as Mr. Herschy lists for the individual ratings. This is a matter of quality control. We have calibrated 10 per cent of our meters which amounted to 227 calibrations this year. We are now analysing these calibrations. The results indicate standard errors of approximately two-thirds the value mentioned here, resulting from tightening of the manufacturing of the current meters.

Garg: In India, we are measuring discharges on large rivers carrying discharges to about 60 000 m^3/sec. Have errors in higher ranges, say for discharges of about 3,000 m^3/sec. occurred and what is the percentage of error?

Herschy: Errors at lower discharges are generally higher than errors at the higher discharges, within reasonable limits of course.

Wartena: The number of verticals must increase with increasing width of the stream. Did you neglect this tendency, which is not linear.

Herschy: The largest component of the error is certainly emerging from the verticals. The number of verticals reduces the errors.

Wartena: I wanted to point out the number of verticals has to increase with the width of the stream, but not proportionally. How did you bring this into calculation? I understood that you neglected this.

Herschy: No, this is included in equation 1. The tables show a number of verticals with the error for these verticals.

Wartena: Yes, but that must depend on the dimension length schedule?

Herschy: Yes.

Relation entre la pulsation et la précision du jaugeage par moulinets

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Résumé. Les jaugeages du débit exécutés dans un canal à section rectangulaire, derrière une grille de tranquillisation, ont démontré que les pulsations de l'écoulement décroissent en fonction de la distance, avec une bonne approximation, suivant une hyperbole. Le débit mesuré aux moulinets et l'erreur relative de cette mesure décroissent de façon linéaire avec la distance. Le débit mesuré aux moulinets croît suivant une parabole, en fonction de l'écart absolu et de l'écart relatif des vitesses. L'erreur relative du jaugeage par moulinets est proportionnelle au carré de l'écart absolu et de l'écart relatif, c'est-à-dire que l'erreur de la mesure est en relation linéaire avec l'énergie de turbulence. Avec un écart relatif de 4 %, l'erreur de la mesure peut atteindre 3 %, dans des conditions données. Par conséquent, il faut choisir la section de mesure de manière que l'écart relatif ne dépasse pas 4 % et employer des moulinets dont l'hélice a une masse aussi réduite que possible.

RELATION BETWEEN PULSATION AND THE ACCURACY OF DISCHARGE MEASUREMENT CARRIED OUT WITH A CURRENT METER

Abstract. Rased on measurements carried out in a square section of the channel, it has been shown that behind the stilling grid the pulsation of the water movement decreases hyperbolically with the distance. Simultaneously, the measured discharge and the relative error of the discharge measurement decreases approximately linearly with the distance. The measured discharge increases parabolically with the average variance and with the relative variance. The relative error of the discharge measurement is proportional to the second power of variance and relative variance, that is, the measurement error is related in linear proportion to the turbulent energy. With 4 per cent average relative variance, the caused error reaches, under given conditions, 3 per cent. Therefore the measuring places must be selected so that the relative variance does not exceed 4 per cent.

RELACIÓN ENTRE LA PULSACIÓN Y LA PRECISIÓN DE LA MEDICIÓN DE CAUDAL REALIZADA CON UN MEDIDOR DE CORRIENTE

Resumen. Se ha comprohado, con base a las mediciones realizadas en un canal de sección rectangular, que detrás de la rejilla de remanso, la pulsación del movimiento del agua decrece hiperbólicamente en función de la distancia. Simultáneamente, el caudal medido y el error relativo de la medición de caudal decrece aproximadamente en forma lineal en función de la distancia. El caudal medido aumenta parabólicamente con la varianza media y con la varianza relativa. El error relativo de la medición de caudal es proporcional a la segunda potencia de la varianza y de la varianza relativa, es decir, el error de medición está en proporción lineal a la energía de turbulencia. Para una varianza relativa media del 4 %, el error originado alcanza, para las condiciones dadas, el 3 %. Por tanto, los lugares de medición se deben seleccionar de tal forma que la varianza relativa no debería exceder del 4 %. Si es posible, se debería aplicar un tornillo de poca masa.

ЗАВИСИМОСТЬ МЕЖДУ ПУЛЬСАЦИЕЙ И ТОЧНОСТЬЮ ИЗМЕРЕНИЯ РАСХОДА ВЕРТУШКОЙ

Резюме. На основании измерений, выполненных в канале прямоугольного сечения, установлено, что за успокоительной решеткой пульсация потока

уменьшается по гиперболическому закону с расстоянием. Одновременно измеренный расход и относительная ошибка измерения расхода уменьшаются приблизительно по линейному закону с расстоянием.

Измеренный расход увеличивается по закону параболы со средней и относительной изменчивостью скоростей (пульсацей). Относительная ошибка измерения расхода пропорциональна второй степени изменчивости скоростей, т. е. ошибка измерений связана линейной зависимостью с энергией турбулентности. При средней относительной изменйивости скоростей, равной 4 %, вызываемая ею ошибка при данных условиях достигает 3 %. Поэтому измерительные створы должны выбираться так, чтобы относительная изменчивость не превышала 4 %. По возможности должен применяться лопастной винт с малой массой.

Une source d'erreurs importante des jaugeages par moulinets hydrométriques est la pulsation des vitesses d'écoulement de l'eau. Ce phénomène n'a que peu d'influence lors du tarage des moulinets par remorquage en eau calme, de sorte que les conditions de tarage et les conditions de mesure sont très différentes. Ces différences causent des erreurs systématiques et accidentelles. Ces dernières peuvent être réduites en consacrant plus de temps aux mesures et en augmentant le nombre des points de mesure. En revanche, les erreurs systématiques ne peuvent être réduites qu'à l'aide du choix judicieux de la section de jaugeage, de la méthode de mesures et des appareils. Il est donc évident qu'il faut connaître la relation entre la pulsation et l'erreur causée, afin de pouvoir déterminer, le cas échéant, la précision des mesures exécutées.

LA METHODE DE L'INVESTIGATION

Dans un canal à section rectangulaire le débit a été mesuré d'une manière absolument semblable en trois sections à pulsations différentes. Simultanément le débit permanent de l'écoulement a été mesuré, avec une précision aussi poussée que possible, à l'aide d'un déversoir type "*Bazin*". Les débits mesurés à l'aide des moulinets ($Q_{\text{mes.}}$) ont été comparés aux débits déterminés au déversoir ($Q_{\text{dév.}}$) et la valeur relative

$$Q\ [\%] = \frac{Q_{\text{mes.}} - Q_{\text{dév.}}}{Q_{\text{dév.}}} \cdot 100$$

a été calculée en fonction des grandeurs caractéristiques de la pulsation, de l'écart absolu D de la vitesse et de l'écart relatif *d*.

Le canal d'essai est un canal en béton d'une longueur de 10 m et d'une largeur de 1 m. A 2 m de l'entrée se trouvait une grille de tranquillisation et à 10 m le déversoir type "*Bazin*". Les jaugeages ont été exécutés dans les sections distantes respectivement de 0,5 m, 5 m, 7 m de la grille (fig. 1*a*).

Les signaux des 7 moulinets "Ott Minor" munis d'hélices de 5 cm de diamètre et au pas de 0,25 m, montés sur une perche verticale, ont été enregistrés par un chronographe pendant 10 minutes. Deux perches semblables ont été placées successivement dans les huit verticales de chaque section pour obtenir les vitesses dans les points de mesure indiqués dans la figure 2. Cette figure montre en outre la distribution des vitesses dans la section médiane. Le débit a été déterminé en multipliant l'aire de la partie de la section de mesure, appartenant au point de mesure en question, par la vitesse moyenne suivant le temps afférent, et en additionnant ces résultats.

Le débit passant par le déversoir a été calculé au moyen de la formule "**Rehbock**", car ce procédé a donné une bonne concordance avec la courbe des débits établie par tarage. Au cours des investigations, le débit calculé de cette manière a été égal à

$$Q_{\text{dév.}} = 1{,}066 \text{ m}^3/\text{s}$$

Les caractéristiques de la pulsation ont été calculées sur la base de 120 valeurs de vitesse déterminées dans des intervalles de 5 secondes sur les bandes du chronographe. La figure 3 montre la distribution des écarts D en cm/s et la figure 4 la distribution des écarts relatifs *d* en % dans la section médiane. Dans chaque section la moyenne arithmétique des écarts a été calculée et la dispersion moyenne à été caractérisée par cette valeur. En divisant l'écart moyen par la vitesse moyenne de la section, on a obtenu l'écart relatif moyen caractérisant cette section. Les vitesses moyennes basées sur les intervalles de 5 secondes s'adaptent bien à la distribution normale. La probabilité *p* % de cette adaptation a été supérieure à 70 % dans les 40,5 % des mesures et inférieures à 30 % dans les 19 % des mesures. Elle n'est jamais tombée au-dessous de 10 %. La figure 5 donne la distribution approximative de l'adaptation dans la section médiane.

L'indépendance des données a été examinée à l'aide de la méthode "**Wald**" et "**Wolfowitz**". La probabilité de l'indépendance a été d'environ 50 %. En divisant, suivant l'ordre chronologique, les données en deux parties égales, l'homogénéité des nouvelles séries a été déterminée. La probabilité de l'homogénéité a été toujours supérieure à 10 % et dans les 80 % des cas, elle a été comprise entre 30 et 70 %. Parmi les données simultanées, la corrélation suivant la position décroît rapidement. Entre les valeurs des vitesses au point de mesure le plus haut et au point plus bas de 0,2 m, le coefficient de corrélation varie entre 0,5 et 0.7. Entre les valeurs des vitesses au point le plus haut et au point plus bas de 0,4 m, le coefficient de corrélation reste inférieur à 0,1.

EVALUATION DES RESULTATS

L'écart moyen et l'écart relatif, caractéristiques des sections, ont été représentés en fonction de la distance (*M*), mesurée à partir de la grille de tranquilisation (fig. 1*b*). Cette relation, c'est-à-dire la variation de la pulsation en fonction de la distance de la grille, est caractérisée par une section d'hyperbole, déterminée graphiquement sur papier en coordonnées logarithmiques. La courbe des écarts relatifs a été tracée de la même manière. Cette relation ressemble à la relation déterminée par MM. **Castex** et Carvounas et basée sur leurs propres mesures [1]. Mais il n'est pas possible d'établir une comparaison numérique entre ces résultats de mesure, car les caractéristiques de pulsation de leurs sections de mesure ne sont pas connues et les masses des hélices des moulinets sont très différentes dans les deux cas.

La figure 1*c* montre l'erreur relative ΔQ en fonction de la distance entre la section et la grille. Les points se rangent sur une ligne droite. C'est également le cas pour les points des débits mesurés (Q_{mes}) [fig. 1*d*].

La valeur de débit $Q_{\text{dév.}}$ est également indiquée. La figure 6*a* montre la relation entre le débit mesuré et l'écart moyen, et entre le débit et l'écart relatif.

La figure 6*b* montre la variation de l'erreur relative en fonction de l'écart moyen et l'écart relatif dans chaque section. Ces résultats donnent, après avoir été rajustés par la méthode graphique sur papier à double graduation logarithmique, une relation

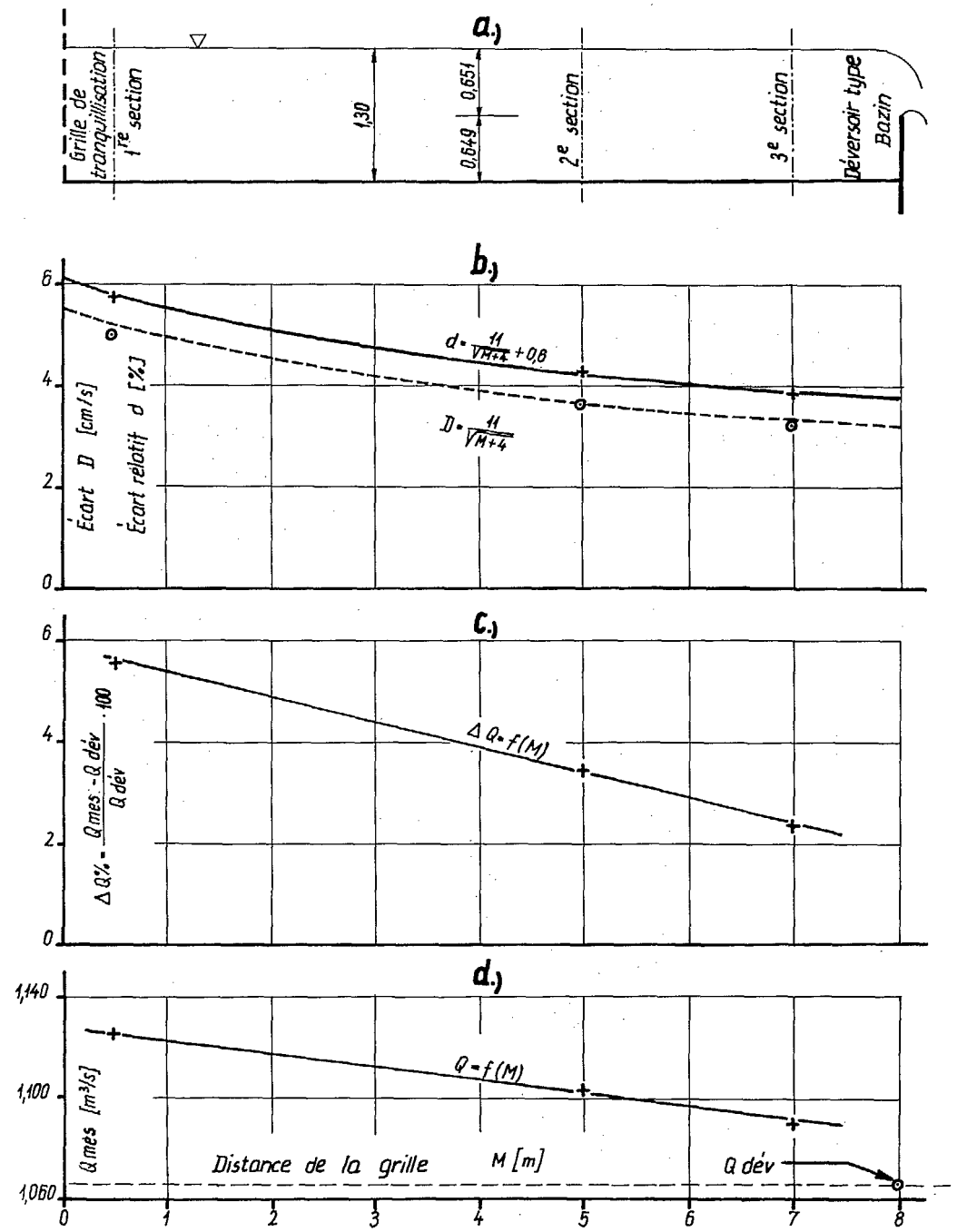


Figure 1. *Les variations de l'écart, de l'écart relatif, de l'erreur relative et du débit d'eau au long du canal.*

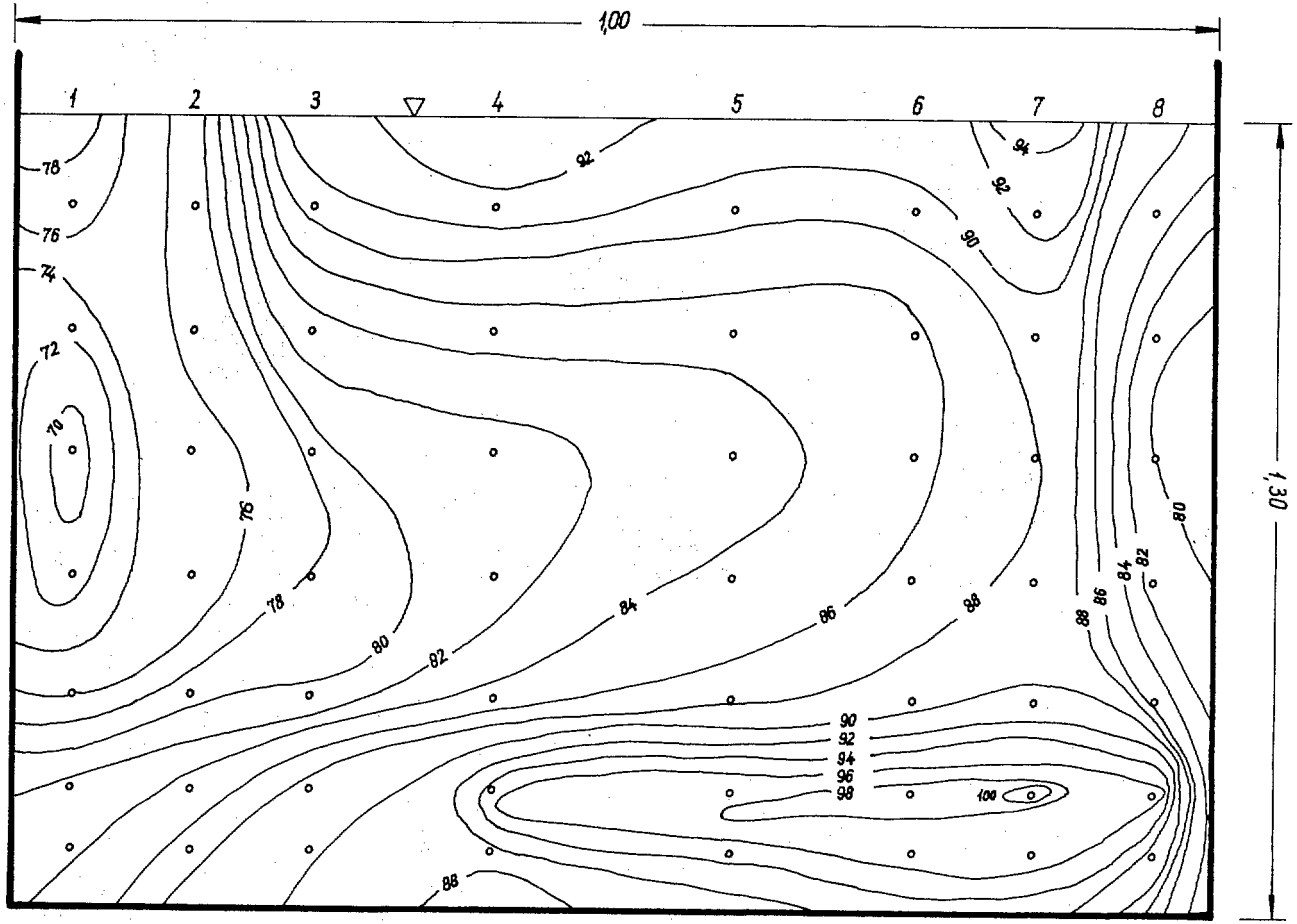


Figure 2. *Distribution des vitesses dans la section 2. Isotaches par 2 cm/s*

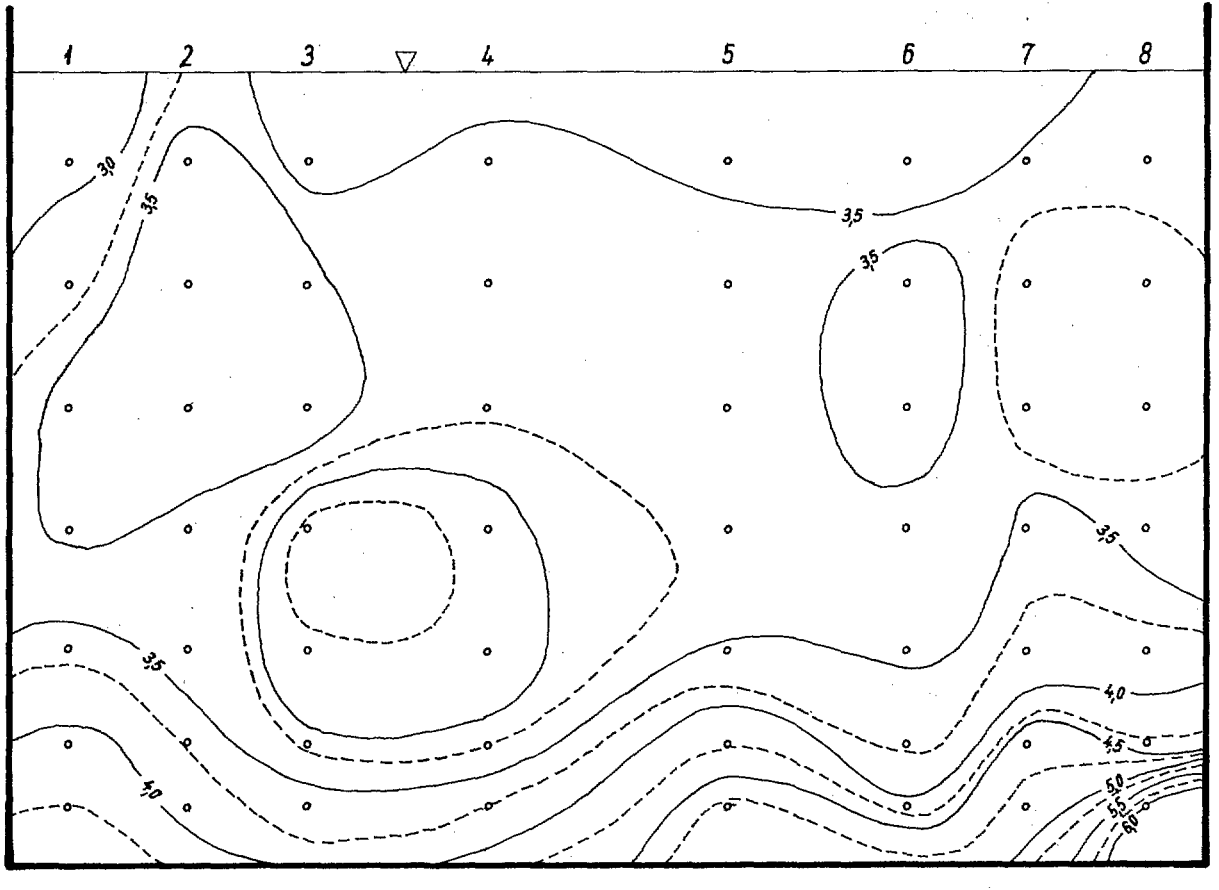


Figure 3. *Distribution des écarts de vitesses dans la section 2. Lignes d'écarts égaux par 0,25 cm/s*

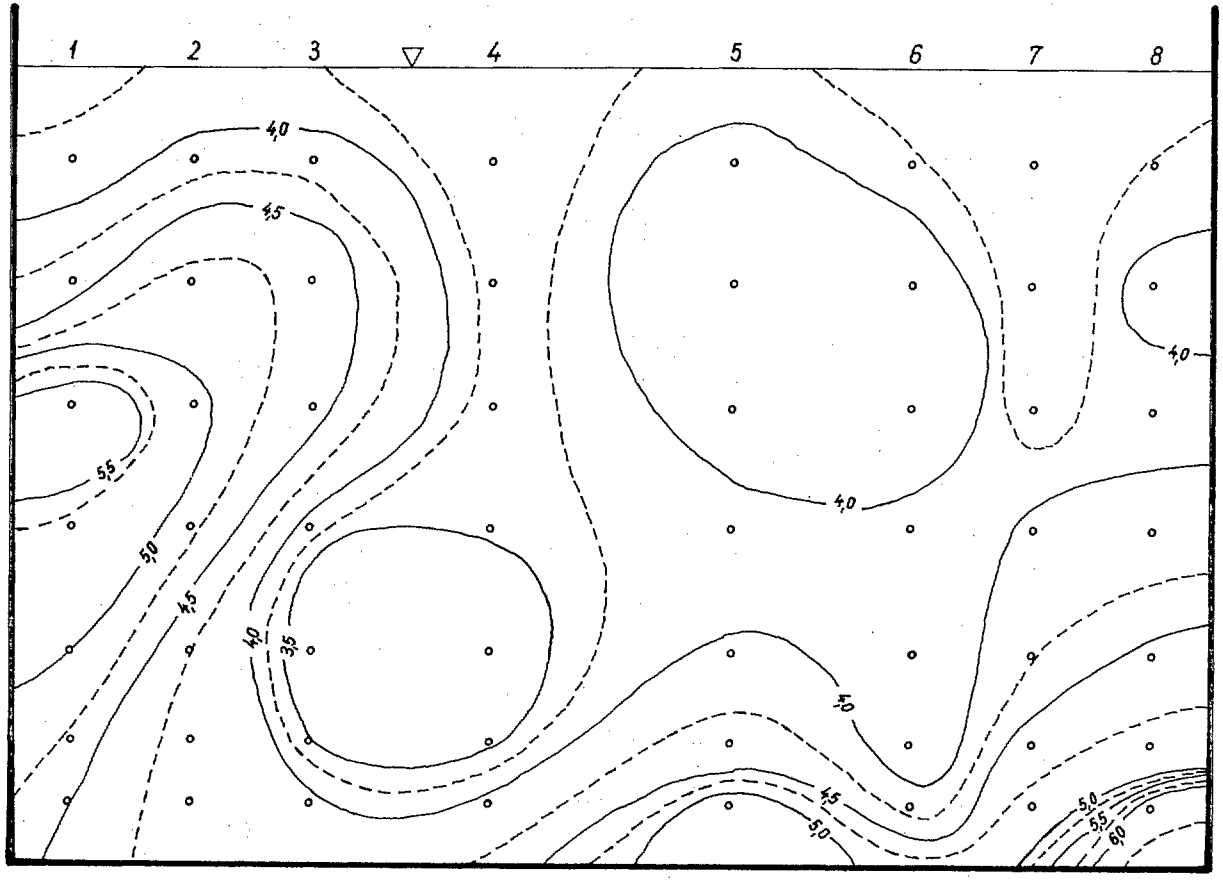


Figure 4. *Distribution des écarts relatifs dans la section 2. Lignes d'écarts égaux par 0,25 %.*

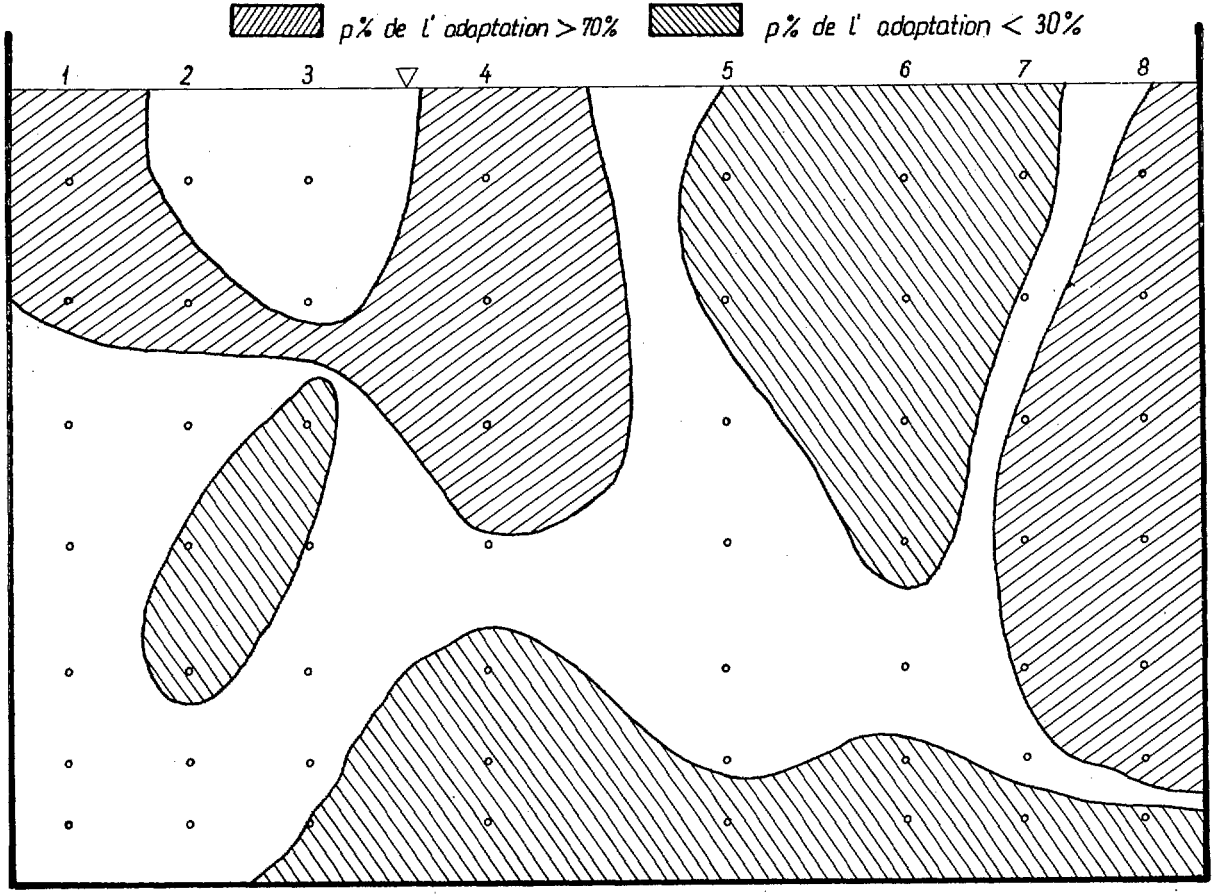


Figure 5. *Distribution de la probabilité de l'adaptation dans la section 2*

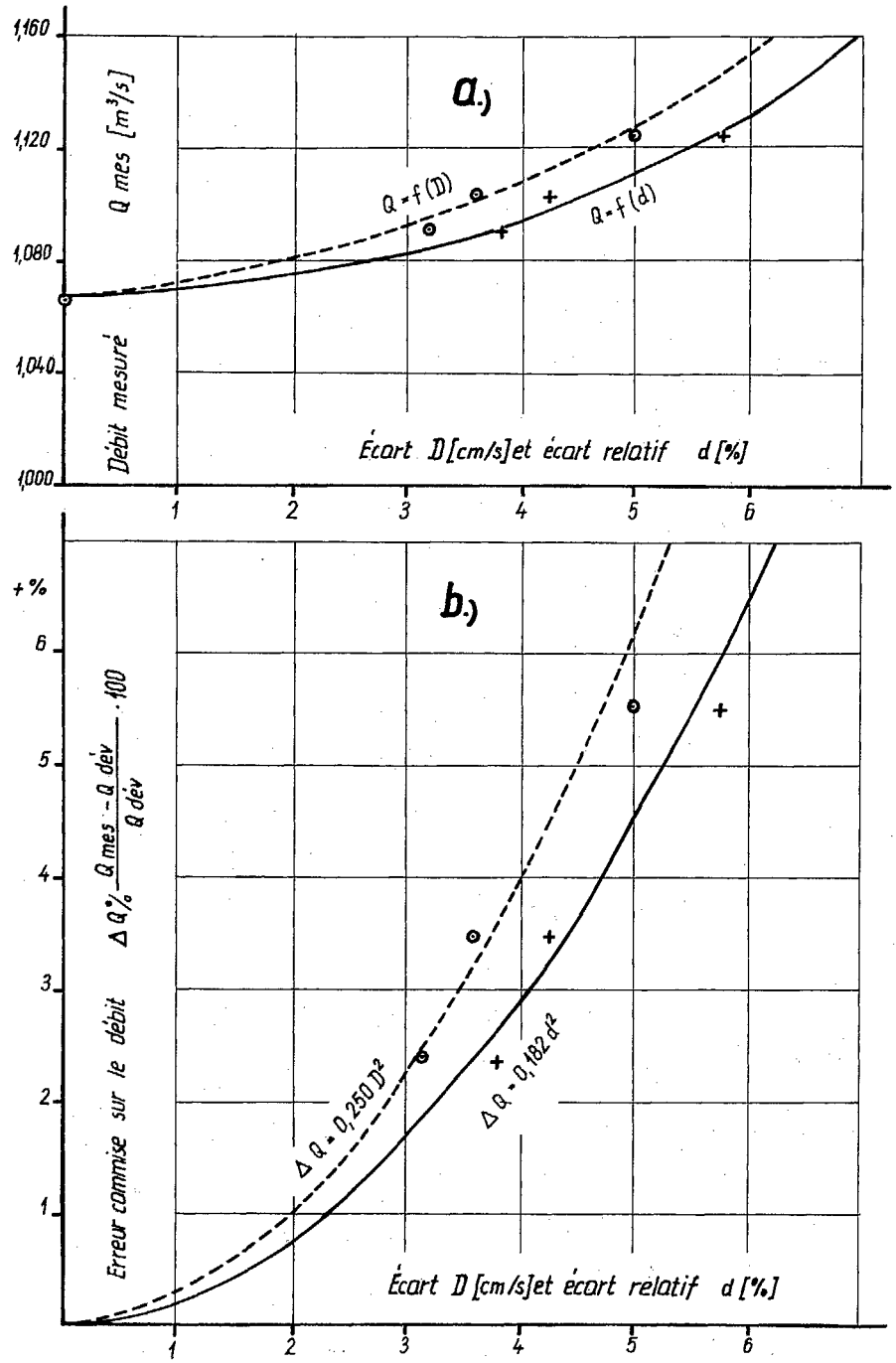


Figure 6. *Débit d'eau et erreur relative en fonction de l'écart absolu et de l'écart relatif des vitesses*

nettement quadratique. Ce résultat est en concordance avec l'interprétation physique des écarts quadratiques des mouvements d'eau, interprétation donnée par M. Szigyártó [2]. On peut démontrer que la moitié de l'écart quadratique des vitesses mesurées dans une certaine direction est égale au surplus présumé de l'énergie cinétique dû aux turbulences, surplus rapporté à la masse unitaire de l'eau en mouvement, soit la valeur présumée de l'énergie de turbulence. Cela veut dire en pratique que le moulinet hydrométrique ne mesure pas seulement la vitesse d'écoulement de l'eau, mais il y ajoute une certaine valeur, en transformant une certaine partie de l'énergie de pulsation en mouvement de rotation. La valeur de ce surplus de vitesse dépend du caractère de la pulsation, de la masse de l'hélice, de sa forme et du frottement dans les roulements [1].

CONCLUSION

En jaugeant les débits aux moulinets hydrométriques, l'erreur due à la pulsation de l'écoulement est en relation étroite avec l'écart moyen des vitesses caractéristiques de la section. L'erreur relative croît avec le carré des valeurs de l'écart. Il est donc recommandé de ne pas mesurer aux moulinets le débit d'un écoulement à forte pulsation. Il est possible de réduire l'erreur en réduisant la masse de l'hélice de l'instrument de mesure. Afin d'arriver à une estimation de l'erreur commise, il faut établir, pour les appareils de mesure employés, des relations semblables à celles développées ci-dessus et déterminer les valeurs des écarts moyens caractéristiques de la pulsation du débit à mesurer.

BIBLIOGRAPHIE/BIBLIOGRAPHY

1. Castex, L.; Carvounas, E. 1962. *Influence de la turbulence sur la mesure du débit en canal à l'écoulement libre au moyen de moulinets.* Glasgow, Kilbride. (ICMG Report n° 13.)
2. Szigyártó, Z. 1962. *Les bases de la théorie statistique de la turbulence.* Budapest, Hydrológiai Közlöny. (En hongrois, avec résumés en russe et en anglais.)

1. Avec un écart relatif de 4 %, l'erreur de la mesure peut atteindre 3 %, dans des conditions données. Par conséquent : choisir la section de mesure de manière que l'écart relatif ne dépasse pas 4 % et employer des moulinets dont l'hélice a une masse aussi réduite que possible.

Instruments for measurement of currents and levels in natural reservoirs and rivers

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Abstract. The distant current recorder type IM-27 is designed for the measurement of current speed and direction to a depth of 300 m. The operating principle of the instrument is based on the transformation of current speed and direction into electric pulses transmitted by cable to the recorder of the device. Special features of the instrument are as follows: the absence of a roll of cable, and twisting of the cable when reading the instrument; the absence of motion when taking readings; and a special pulley which permits the automatic measurement of discrete current directions and gives the mean direction every three minutes. Because of this, the device is very operational and easily used.

The hydrostatic level sensor with a pneumatic communication line is designed for measuring the level of natural reservoirs and rivers which form part of a system of automatic hydrometeorological stations. The operating principle is based on the transformation of hydrostatic pressure into pneumatic pressure by means of a power compensation system, the output being shown on the potentiometer. Its distinctive features are the high level of accuracy, with scales ranging from 1 m to 12 m, and the all-metal construction of the pneumatic system which ensures high sensor reliability and stability.

APPAREIL DE MESURES DES COURANTS ET DES NIVEAUX DANS LES BASSINS NATURELS ET LES RIVIERES

Résumé. L'enregistreur de télécommande de courant IM-27 est destiné à mesurer la vitesse et la direction des courants à des profondeurs allant jusqu'à 300 mètres. Le principe de fonctionnement de cet appareil repose sur la transformation de la vitesse et de la direction du courant en un nombre d'impulsions électriques transmises à l'aide d'un câble à la partie enregistreuse de ce dispositif. Les particularités de cet appareil sont les suivantes : ses indications ne dépendent pas de la torsion du câble, mais en partie de l'agitation ; l'utilisation d'une poulie spéciale permet l'enregistrement automatique de toutes les directions de courant discontinues mesurées, et donne la direction moyenne dans l'intervalle de temps de mesure de 3 minutes. Cette particularité rend ce dispositif très fonctionnel et d'une utilisation facile. Le capteur hydrostatique de niveau pneumatique est destiné à la mesure du niveau des bassins naturels et des rivières dans le système des stations automatiques hydrométéorologiques. Son principe de fonctionnement est basé sur la transformation de la pression hydrostatique en pression pneumatique avec mesure à l'aide d'un système de compensation de force et sortie sur un potentiomètre.

Cet appareil est caractérisé par une précision élevée – pour une variation de l'échelle de 1 à 12 mètres – par une construction entièrement métallique du système pneumatique assurant une haute fidélité et par la stabilité du capteur.

INSTRUMENTOS PARA LA MEDICIÓN DE CORRIENTE Y NIVEL EN EMBALSES NATURALES Y RÍOS

Resumen. El registrador de corriente a distancia tipo IM-C7 está proyecado para la medición de la velocidad de la corriente y su dirección a una profundidad de 300 m. El principio de funcionamiento del instrumento se basa en la transformación de la velocidad y dirección de la corriente en impulsos eléctricos transmitidos por cable al registrador del dispositivo.

Las características especiales del instrumento son las indicadas a continuación:

Independencia de la lectura del instrumento con el rollo de cable, una pequeña dependencia de la lectura de la excitación, y también la disponibilidad de una polea especial que permite la consideración de todas las direcciones discretas de corriente medidas automáticamente, que da la dirección media por intervalos de tiempo de 3 minutos. Gracias a esta característica, el dispositivo es muy funcional y el manejo cómodo.

El detector de nivel hidrostático con línea de comunicación neumatica esta proyectado para medir el nivel en los embalses naturales y ríos en el sistema de estaciones hidrometeorológicas automáticas. El principio de funcionamiento se basa en la transformación de la presión hidrostática en neumática, con la ayuda de un sistema de compensación de potencia en la salida de un potenciómetro.

Las características distintivas son una elevada precisión, versatilidad de la escala, desde 1 m a 12 m, y la construcción totalmente metálica del sistema neumático, que garantiza una elevada fiabilidad y estabilidad del detector.

ПРИБОРЫ ДЛЯ ИЗМЕРЕНИЯ ТЕЧЕНИЙ И УРОВНЯ В ЕСТЕСТВЕННЫХ ВОДОЕМАХ И РЕКАХ

Резюме. Дистанционный самописец течений ГМ-27 предназначен для измерения скорости и направления течений на глубинах до 300 м. Принцип действия прибора основан на преобразовании скорости и направления течений в электрические импульсы, посылаемые по кабелю в регистрирующую часть прибора.

Отличительными особенностями прибора являются: независимость его показаний от скручивания кабеля, малая зависимость показаний от волнения, а также наличие специального блока, позволяющего автоматически учесть все измеренные дискретные направления течения и выдать среднее направление за трехминутный интервал измерения. Эта особенность делает прибор очень оперативным и удобным в эксплуатации.

Гидростатический датчик уровня с пневматической линией связи предназначается для измедения уровня в естественных водоемах и реках в системе автоматических гидрометеорологических станций. Принцип работы основан на пребразовании гидростатического давления в пневнатическое с последчющим его измерением при помощи системы силовой компенсации с выходом на потенциометр.

Отличительными особенностями вяляются высокая точность, сменность масштаба от 1 м до 12 м, а также цельнометаллическая конструкция пневматической системы, обеспечивающая высокую надежность и стабильность датчика.

A remote-indicating instrument for effective measurements of current speed and direction has been developed in the U.S.S.R. This device permits current measurements to be made in rivers, storage reservoirs and lakes at depths of up to 300 m.

The instrument comprises two main units (see Fig. 1), connected to each other by a five-conductor cable; these are the current sensor, submerged in the water, and the recorder which is installed on the deck of a vessel.

Magnetic couplings are used in the current sensor, therefore cable twisting cannot adversely affect the results of measurements. This feature of the design allows the accuracy of measurement to be raised and simplifies the handling of the instrument, because the current sensor may be lowered directly on a cable or a cable-laid rope without a swivel.

The values of current speed and direction are converted into electric pulses for remote data transmission. These pulses are produced by means of both photoresistors which change their electric resistance under light, and an induction coil whose inductive reactance depends on the position of its movable iron core.

Rotation of the Savonius-type rotor of the current meter is imparted through the magnetic couples to a slotted disc attached inside the case of the current sensor. The number of disc revolutions is converted by means of a photo resistor and a flashlight electric lamp into electric pulses which are transmitted via the cable to the deck recorder. In the recorder these pulses are converted by an electromechanical system into the angular displacement of a marking striker attached to a special reduction gear.

The average current speed is recorded over 3-minute periods of measurement.

Thus, the higher the current speed, the more electric pulses will be produced and the greater will be the angular displacement of the marking striker.

The current direction is expressed by the angle between the direction of a vane and the magnetic meridian. For determination of this angle two synchronous step motors are installed in the current sensor and the recorder. The first of them—the master motor installed inside the current sensor—has a mirror and a spiral cam on its shaft. By means of a light beam reflected from this mirror, the master motor searches for the slot in an opaque compass card. The direction of this slot always coincides with the magnetic meridian.

By means of the spiral cam, the master motor finds the direction of the vane which is oriented in the direction of the current. As soon as these two directions (i.e. those of the magnetic meridian and the vane) have been determined by the master motor, electric pulses are sent to the recorder. The pulses cause the striker to plot appropriate marks on a paper strip chart. The chart drum rotates synchronously with the master motor, being driven by the second step motor.

The recorder is provided with a unit for automatic averaging of the current direction. In this unit the mean value of the angles being measured is defined by the number of electric pulses divided by the number of measurements taken. Horizontal and vertical graduation lines are printed on the paper chart, corresponding to standard depths, current speed and current direction (see Fig. 2).

Up to eight series of data obtained over the 3-minute cycle of current measurement can be recorded on one chart. These measurements may be taken at various depths, but to simplify the data processing the scale of standard depths is printed on the chart. The actual depth of the current sensor submersion is monitored by a standard meter wheel (pulley-counter).

Each separate mark left by the striker on the chart in the form of a *red* point (see "3" in Fig. 2) corresponds to a mean value of the current speed recorded over a 3-minute period of measurement. A series of *black marks* ("2" in Fig. 2) show discrete values of the current direction.

No less than eleven instantaneous measurements of the current direction are taken over the 3-minute period, and the recorder calculates automatically the mean direction from ten of these instantaneous values.

Since it is necessary to measure the angle between the current direction and the magnetic meridian to calculate the current direction, there is, in addition to the marks labelled "2", a series of marks shown at "1" on Fig. 2, and the angle between the marks "2" and "1", expressed in degrees represents the current direction. The instrument is designed in such a way that in normal operation the marks "1" always coincide with the base line so that the marks "2" plotted on the chart represent the current direction. Thus, the entire current profile will be shown visually on the paper chart.

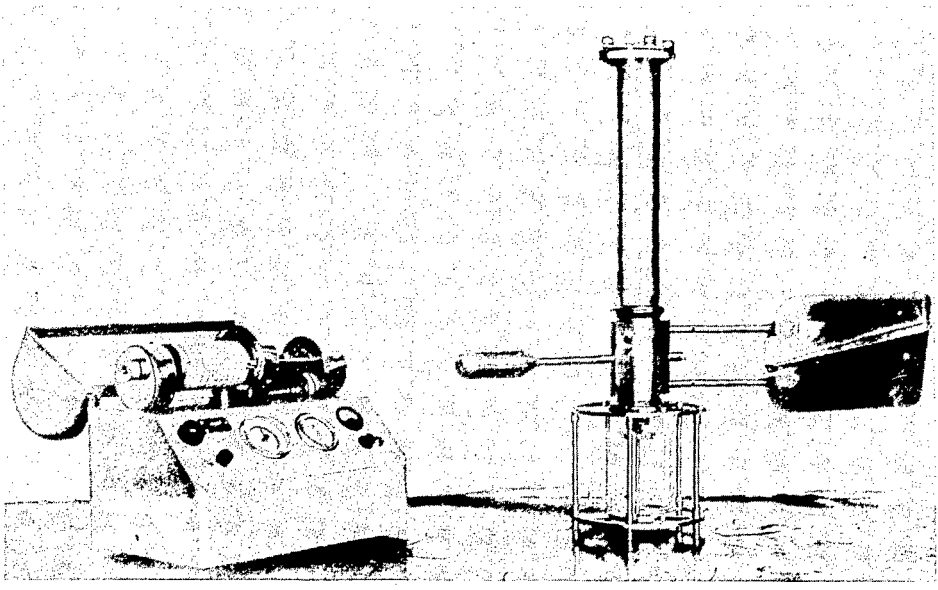


Figure 1. *The remote-controlled current recorder*

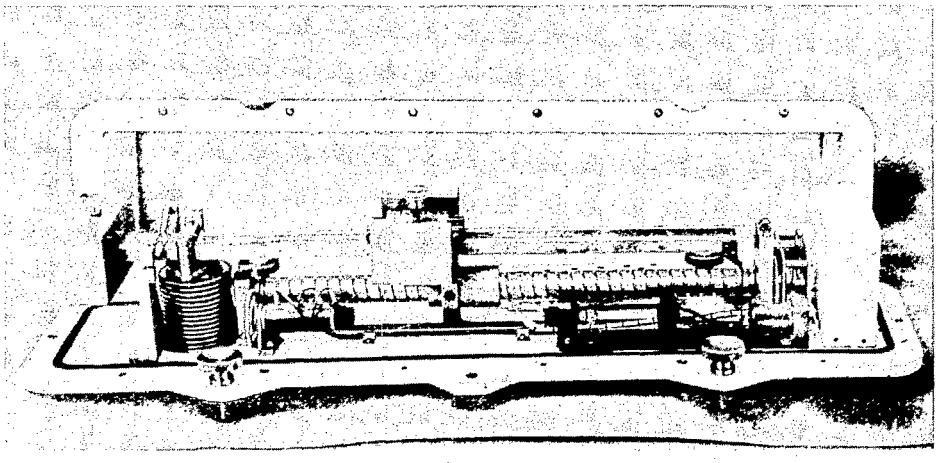


Figure 3. *The version of the pneumatic level gauge measuring unit destined for automatic hydrometeorological stations*

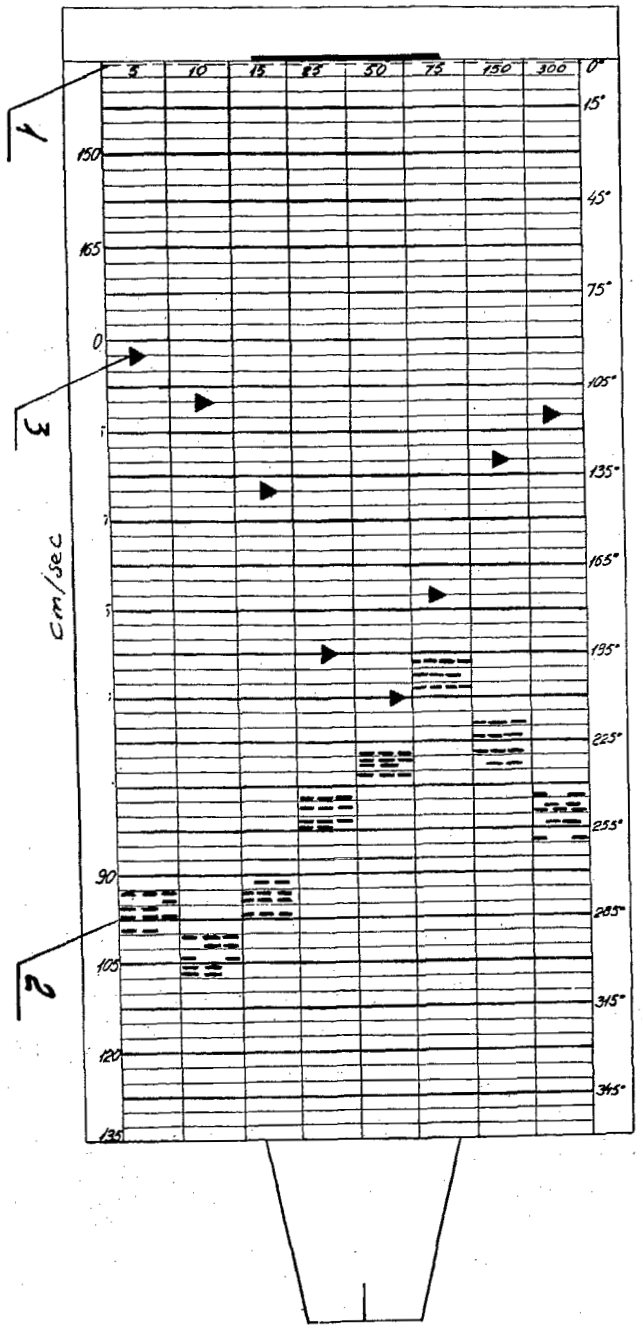


Figure 2. *Pattern of the chart of the current recorder*

A plot of changes of both current direction and speed at various depths may be drawn by connecting all these marks. In this case, the points representing values of the current speed can be connected together by a line without much trouble, because every depth will be represented here with only one point which corresponds to the mean current speed. For plotting the current direction curve one has to determine the average current direction for this depth over the 3-minute period of measurement.

The main difficulty in measuring the mean current direction, in contrast to measurements of the mean current speed, lies in the fact that the current direction value can fluctuate on both sides of the base line, and therefore positive pulses as well as negative ones must be taken into account for more precise determination of the mean current direction. In order to eliminate any possibility of fluctuation of the direction values on both sides of the base line and, consequently, to avoid complex measurements and the summation of positive and negative angles, the instrument has been designed so that the mean current direction is measured from a reference direction for the given cycle of measurement. This reference direction is selected automatically by the instrument itself to be opposite the first discrete value of the current; for example, if the current is 90°, the instrument will automatically select the reciprocal as the reference direction, i.e. 270°. Over the 3-minute period of measurement, the current direction will not vary by more than 180° as compared with the first discrete value recorded; consequently, all the angles measured from the reference direction will be positive, and they may therefore be calculated by means of a simple electromechanical computer incorporated in the recorder. The mean current direction is thereby determined with the help of this computer, two calibrated dials and a pointer. One of the dials is movable, and at the end of the measurement cycle it must be set in accordance with the reference direction, i.e. this dial takes account of the selected reference direction.

This instrument seems to be the first remote-controlled recorder which automatically calculates the mean current direction from separate discrete values recorded on the paper chart.

Measurement range:	
Current speed	3-360 cm/sec
Current direction	0-360°
Accuracy of speed measurement	± (2 cm/sec + 3 per cent value measured)
Starting threshold of current-speed sensor	2 cm/sec
Accuracy of direction measurement	± 5°
Operation depth	up to 300 m
Weight of current sensor	10 kg
Weight of recorder	20 kg
Over-all dimensions of the current sensor:	
Height	680 mm
Length (with vane)	630 mm
Diameter of case	55 mm
Diameter of rotor shield	170 mm
Over-all dimensions of recorder	290 x 330 x 210 mm

For water-level measurements in open basins and rivers, a pneumatic gauge has been designed. In this instrument hydrostatic pressure is converted into air pressure which is measured by means of a special force-compensating system provided with a positively controlled weight moving along a balancing lever.

Figure 3 shows one version of a gauge measuring unit. This device is destined for use in an automatic hydrometeorological station. The pressure of air contained in a pneumatic system is measured by means of bellows, balancing lever, movable weight, worm shaft, null (balance) detector, reduction gear and reversible electric motor.

A precision multiturn potentiometer permits the use of this measuring unit as a water-level transducer for automatic hydrometeorological stations, and in addition, the measuring unit is provided with a clockwork chart drum for permanent recording of water levels.

The entire pneumatic system is made of metal and therefore it can be hermetically sealed. In spite of the all-metal pressure sensor, the instrument is free from temperature errors; moreover, it is quite versatile, since it permits the water level to be measured on various scales and in any range from 0 to 12 m (depending upon the changeable weight used).

Accuracy of hydrostatic pressure measurement	± 0.1 per cent of full range
Over-all dimensions of pressure sensor	dia. 300 × 600 mm
Over-all dimensions of measuring unit	180 × 180 × 550 mm
Weight of pressure sensor	not more than 10 kg
Weight of measuring unit	not more than 20 kg.

Flow measurement of some of the world's major rivers by the moving-boat method

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Abstract. A method for the rapid measurement of flows of large rivers and tidal reaches is presented. Theory of the technique is developed and equipment utilized is described. In this method, data are collected at a number of observation points while the observer is aboard a small boat that is rapidly traversing the cross-section. This technique is particularly useful at remote sites where no measuring facilities exist, during floods when facilities may be inundated or inaccessible, or at locations where unsteady flow conditions require rapid measurements.

Flow measurements made at various sites using the moving-boat technique are summarized. Included are results obtained on two tidal reaches and such major rivers of the world as the Amazon, Mississippi, and Mekong. Where information is available, these measurements are compared with concurrent measurements obtained by conventional methods and agreement is generally within 5 per cent.

MESURE DU DEBIT DE QUELQUES GRANDS FLEUVES DU MONDE PAR LA METHODE DU BATEAU MOBILE

Résumé. Cet article présente une méthode pour la mesure rapide des débits des grandes rivières et des biefs à marée. Il développe la théorie de cette technique et décrit l'appareillage utilisé. Dans cette méthode, les données sont recueillies aux différents points pendant que l'observateur est à bord d'un petit bateau traversant rapidement le cours d'eau. La technique du bateau mobile est particulièrement utile dans les endroits éloignés où les instruments nécessaires aux mesures du débit n'existent pas, pendant les inondations, quand les instruments sont soit inondés, soit inaccessibles, ou encore dans les endroits où les conditions de variation du débit nécessitent des mesures aussi rapides que possible.

Les mesures du débit obtenues dans différents endroits pendant les essais sont récapitulées. Les résultats des mesures sur plusieurs biefs à marée et sur un certain nombre des plus importantes rivières du monde figurent dans la récapitulation. Là où l'information existe, ces mesures sont comparées avec celles obtenues par des méthodes conventionnelles.

MEDICIÓN DE CAUDAL EN UNO DE LOS MAYORES RÍOS DEL MUNDO POR EL MÉTODO DE BARCA MÓVIL [1]

Resumen. Se presenta un método para la determinación rápida de caudales en los grandes ríos y se desarrolla la teoría de esta técnica y describe el equipo utilizado. En este método, se recogen datos en cierto número de puntos de observación, mientras que el observador se sitúa en una pequeña barca con la que atraviesa rápidamente la sección transversal. Esta técnica es particularmente útil en lugares de difícil acceso en los que no existen instalaciones de medida, durante las crecidas, cuando las instalaciones pueden encontrarse inundadas o inaccesibles, o en lugares en que las condiciones de caudal discontinuo exigen una medición rápida.

Se resumen las mediciones de caudal realizadas en diversos emplazamientos utilizando la técnica de la barca móvil. Se incluyen los resultados obtenidos en dos rías y en algunos de los mayores ríos del mundo, como el Amazonas, el Mississipi y el Mekong. En los casos en los que se dispone de la información necesaria, se comparan estas mediciones con las obtenidas por métodos convencionales, y las diferencias por lo general, están comprendidas dentro del 5 %.

ИЗМЕРЕНИЕ РАСХОДА НА НЕКОТОРЫХ КРУПНЫХ РЕКАХ МИРА СПОСОБОМ ДВИЖУЩЕГОСЯ СУДНА

Резюме. Предложен способ для быстрого измерения расходов больших рек и приливных участков. Приводятся некоторые теоретические соображения и описывается используемое оборудование. В этом методе данные собираются со множества наблюдательных точек, в то время как наблюдатель находится в небольшой лодке, которая быстро пересекает поперечное сечение реки. Эта методика особенно полезна на удаленных от населенных пунктов местах, где не имеется измерительного оборудования, во время паводков, когда оборудование может быть затоплено или недоступно, или на участках, где условия неустановившегося движения требуют быстрых измерений.

Обобщены измерения расходов, выполненные в различных местах с использованием движущейся лодки. Включены результаты, полученные на двух приливных участках итаких крупных реках мира как Амазонка, Миссисипи и Меконг. Там, где это было возможно, эти измерения сравнивались с параллельными измерениями, полученными общепринятыми методами, причем совпадение обычно находилось в пределах 5 %.

INTRODUCTION

Conventional techniques for measuring the discharge of streams by current meter are standardized and well known. Frequently, however, on large streams and estuaries standard current-meter methods of measuring discharge are impracticable and involve costly and tedious procedures. This is particularly true during floods, when facilities may be inundated or inaccessible, at remote sites where no facilities exist, or at locations where unsteady flow conditions require that measurements be made as rapidly as possible. The moving-boat technique (Smoot and Novak, 1969) is a method applicable to the rapid measurement of large streams. It requires no fixed facilities and it lends itself to the use of alternate sites if conditions make this desirable.

MOVING-BOAT METHOD OF MEASURING DISCHARGE

The moving-boat technique is similar to the conventional current-meter measurement in that both use the velocity-area approach of determining discharge. In each method, a measurement is the summation of the products of the partial areas of the stream cross-section and their respective average velocities. Both techniques require that the following information be obtained:

1. Location of observation points.
2. Stream depth, d, at each observation point.
3. Stream velocity, V, perpendicular to the cross-section at each observation point.

During a traverse of the boat across the stream, a sonic sounder records the geometry of the cross-section, and a continuously operating current meter senses the combined stream and boat velocities (Fig. 1). A vertical vane aligns itself in a direction parallel to movement of water past it, and an angle indicator attached to the vane assembly indicates the angle between the direction of the vane and the true course of the boat. The data from these instruments provide the information necessary for computing the discharge for the cross-section. Normally, data are collected at 30 to 40 observation points in the cross-section for each run.

The principal difference between the conventional measurement and the moving-boat measurement lies in the method of data collection. The standard current-meter method of measurement uses what might be called a static approach in its manner of sampling; that is, the data are collected at each observation point in the cross-section while the observer is in a stationary position. This is in contrast to the dynamic approach to data collection utilized in the moving-boat method. Here, data are collected at each observation point while the observer is aboard a boat that is rapidly traversing the cross-section.

The traverse is made without stopping along a preselected path that is normal to the streamflow, and the data are collected at regular intervals along the path. During the traverse of the cross-section, the boat operator maintains course by "crabbing" into the direction of the flow sufficiently to remain on line. The velocity, V_b, of the boat with respect to the streambed along the selected cross-section path is the velocity at which the current meter is being pushed through the water by the boat. The force exerted on the current meter, then, is a combination of two forces acting simultaneously; one force resulting from the movement of the boat through the water along the cross-section path, and the other a consequence of the natural streamflow normal to the path.

The velocity measurement taken at each of the sampling points in the cross-section is a vector quantity which represents the relative velocity of water past the vane and meter. This velocity, V_V, is the vector sum of V, the component of stream velocity normal to the cross-section at the sampling point, and V_b, the velocity of the boat with respect to the streambed along the selected path.

The sampling data recorded at each observation point provide the necessary information to define V_V. The pulses-per-second reading from the rate indicator unit is used in conjunction with a rating table to obtain the vector magnitude, V_V, while the angle reading, α, representing the angle the vane makes with the cross-section path, defines the direction of the vector.

Stream velocity, V, perpendicular to the boat path (true course) at each sampling point can be determined from the relationship

$$V = V_V \sin \alpha. \tag{1}$$

The solution of the above equation yields an answer that represents the component of the stream velocity that is perpendicular to the true course, even though the direction of flow may not be perpendicular. This is the desired component.

From the same vector diagram, it can be seen that

$$L_b = \int V_V \cos \alpha \, dt, \tag{2}$$

where L_b is the distance that the boat has travelled along the true course between two consecutive observation points, provided that the stream velocity is perpendicular to the path. If one assumes that α is approximately uniform over the relatively short distance which makes up any one increment, then it may be treated as constant. Therefore, equation 2 becomes

$$L_b \approx \cos \alpha \int V_V \, dt. \tag{3}$$

Now

$$\int V_V \, dt = L_V \tag{4}$$

where L_V is the relative distance through the water between two consecutive observation points as represented by the output from the rate indicator and counter.

Therefore,

$$L_b \approx L_V \cos \alpha. \tag{5}$$

Finally, d, stream depth at each observation point, is obtained from the sonic-sounder chart. Upon determining V, L_b, and d for each vertical, the midsection method of computing a discharge measurement is used.

Much of the accuracy of a moving-boat discharge measurement depends upon the skill of the boat operator in maintaining a true course. Although even the most experienced pilot cannot be expected to keep the boat absolutely on course for an entire run, it is still extremely important that the measurement begin and end on line, and that any deviations from the true course be kept as few in number and as small in magnitude as possible.

During a moving-boat discharge measurement, the current meter is set at a predetermined, fixed depth of from 3 to 4 feet (about 1 m) below the water surface; thus, this technique uses the subsurface method of measuring velocity. The measurement is computed by using constant-depth subsurface velocity observations without adjustment coefficients as though each were a mean in the vertical. In the use of this method, each measured velocity should ideally be multiplied by a coefficient to adjust it to the mean velocity in its vertical. However, it is assumed that in the larger streams where the moving-boat technique be applicable, these coefficients would be fairly uniform across a section, thus permitting the use of an average cross-section coefficient to be applied to the total discharge. Information obtained from several vertical-velocity curves, well distributed across the measuring site, would be needed to determine a representative coefficient for the total cross-section.

It is recommended that the results of at least six individual runs, each with from 30 to 40 observation points, be averaged to obtain the discharge in uniform flow conditions. This is practicable because of the ease and speed with which the extra runs can be made. For unsteady flow conditions on tidal streams, it will usually be desirable not to average the results from a series of runs, but rather to keep them separate so as to better define the discharge cycle.

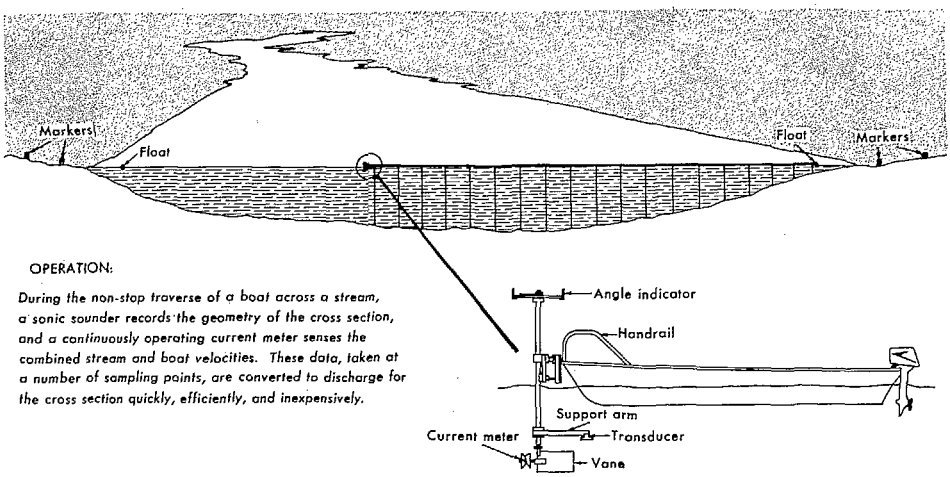
MEASUREMENT RESULTS

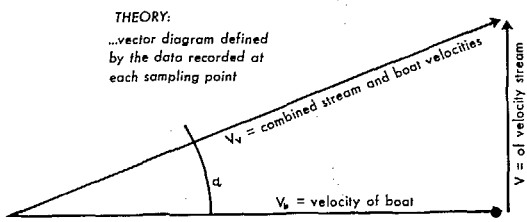
Figures 2-5 and Tables 1-5 present the results obtained using the moving-boat method on several tidal reaches and a number of major rivers around the world. Although space does not permit inclusion of all measurements, those presented are representative of the results that have been obtained using this technique.

Figure 2 is a comparison of moving-boat measurements and conventional current-meter measurements on the Hudson river at Poughkeepsie, N.Y., made over a tidal cycle. The conventional measurements were made with a battery of 21 current meters. Agreement was good at all discharges, including the regions near zero net flow, at which the most discrepancy could be expected.

Table 1 lists the results of a series of moving-boat measurements made on the Alabama river at Selma, Ala. Flow conditions were steady over this period of time, and the average of the 19 moving-boat measurements is within one per cent of the 10,800 cfs (300 m^3/s) discharge as determined from the stage-discharge rating curve.

Table 2 lists the results of a series of 23 moving-boat measurements made on the Mississippi river at Vicksburg, Miss. No single measurement deviates from the average by more than 5.9 per cent.





VELOCITY (V)	×	AREA OF SUBSECTION		
STREAM VELOCITY, V, is computed by the formula $V = V_v \sin \alpha$, where V_v is the velocity of the vane and meter and α is the observed angle.		DISTANCE BETWEEN SAMPLING POINTS, L_b, is computed by the relationship: $L_b = \int V_v \cos \alpha dt$	STREAM DEPTH is obtained from the sonic sounder chart.	DISCHARGE, Q, is the summation of the products of subsection areas and velocities: $Q = \Sigma(aV)$

Figure 1. *Illustration of moving-boat technique and theory*

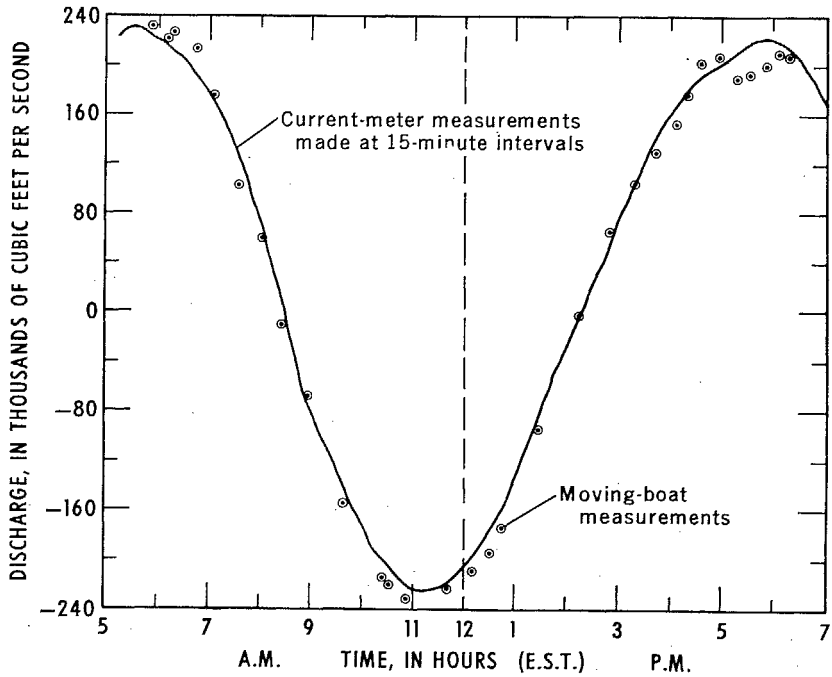


Figure 2. *Discharge hydrograph, Hudson river at Poughkeepsie, N.Y., 30 August 1966*

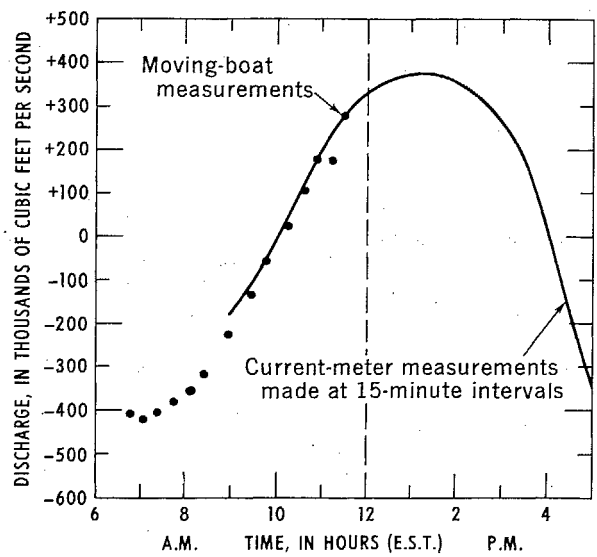


Figure 3. *Discharge hydrograph, Delaware river at Delaware Memorial Bridge, 2 August 1967*

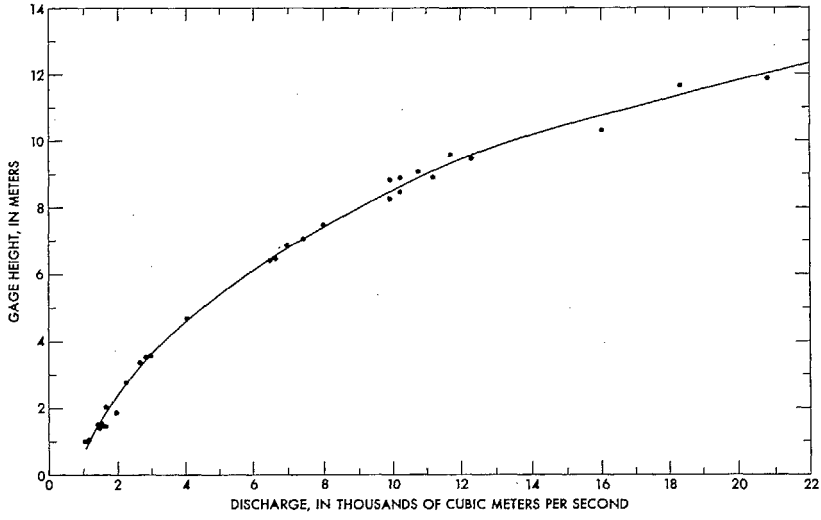


Figure 4. *Rating curve for Mekong river near Vientiane, Laos*

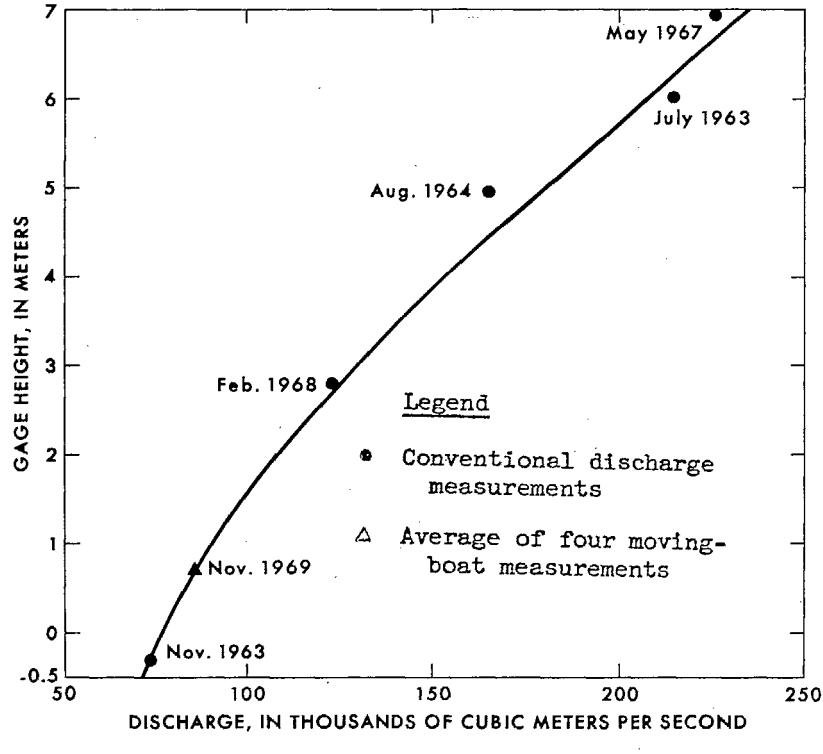


Figure 5. *Plot of measurements made on Amazon river at Obidos, Brazil*

Table 1. *Results of moving-boat measurements on Alabama river at Selma, Ala., 4 October 1966.*

Measurement number	Area in square feet	Discharge in cubic feet per second	Percentage deviation from rating curve discharge
1	4 960	10 400	– 3.7
2	4 940	10 000	– 7.4
3	4 920	10 200	– 5.6
4	4 980	10 600	– 1.9
5	5 080	10 900	0.9
6	5 030	10 900	0.9
7	4 940	10 300	– 4.6
8	4920	10 500	– 2.8
9	4 900	10 300	– 4.6
10	4 900	10 400	– 3.7
11	4 920	11 000	1.9
12	4 830	10 800	0
13	4 820	10 600	– 1.9
14	4 860	10 900	0.9
15	4 860	10 800	0
16	4 870	10 500	– 2.8
17	4 870	10 700	– 0.9
18	4 880	11 200	3.7
19	4 840	11 400	5.6
Average :	4 910	10 700	– 0.9

Table 2. *Results of moving-boat measurements on Mississippi river at Vicksburg, Miss., 6 October 1966.*

Measurement number	Area in square feet	Discharge in cubic feet per second	Percentage deviation from average
1	71 400	257 000	0.4
2	70 000	256 000	0
3	69 400	249 000	– 2.7
4	69 700	249 000	– 2.7
5	70 000	248 000	– 3.1
6	70 200	252 000	– 1.6
7	69 200	268 000	4.7
8	69 200	266 000	3.9
9	68 800	264 000	3.1
10	68 600	256 000	0
11	69 400	253 000	– 1.2
12	69 600	258 000	0.8
13	69 400	254 000	– 0.8
14	69 300	246 000	– 3.9
15	69 600	242 000	– 5.5
16	69 400	254 000	– 0.8
17	69 000	260 000	1.6
18	68 600	265 000	3.5
19	69 000	255 000	– 0.4
20	69 400	267 000	4.3
21	69 000	271 000	5.9
22	68 600	251 000	– 2.0
23	69 600	245 000	– 4.3
Average :	69 400	256 000	

Table 3. *Results of moving-boat measurements on the Mississippi river at St. Louis, Missouri, 20 October, 1967*

Measurement number	Area in square feet	Discharge in cubic feet per second	Percentage deviation from rating curve discharge
1	44 400	148 000	– 5.7
2	44 600	156 000	– 0.6
3	43 600	161 000	2.5
4	44 300	155 000	– 1.3
5	44 200	163 000	3.8
6	44 500	154 000	– 1.9
7	43 500	155 000	– 0.6
8	44 100	156 000	0
9	44 100	145 000	– 6.5
10	44 000	151 000	– 2.6
11	43 100	151 000	– 2.6
12	44 000	151 000	– 2.6
13	43 200	157 000	1.9
14	44 100	152 000	– 1.3
Average :	44 000	154 000	– 1.2

Table 4. *Results of moving-boat measurements on the São Francisco river at Pirapora, Brazil, 19 November 1969*

Measurement number	Area in square metres	Discharge in cubic metres per second	Percentage deviation from rating curve discharge
1	1 290	662	5.1
2	1 270	608	– 3.5
3	1 280	627	– 0.5
4	1 270	616	– 2.2
5	1 280	635	0.8
6	1 270	638	1.3
7	1 270	658	4.4
8	1 280	674	7.0
9	1 280	629	– 0.2
10	1 280	622	– 1.3
11	1 280	611	– 3.0
Average :	1 280	635	0.8

Table 5. *Results of moving-boat measurements on the Amazon river at Obidos, Brazil, 27 November 1969.*

Measurement number	Area in square metres	Discharge in cubic metres per second	Percentage deviation from average
1	101 000	89 200	4.2
2	102 000	87 600	2.3
3	103 000	82 500	– 3.6
4	101 000	83 000	– 3.0
Average :	102 000	85 600	

Figure 3 is a comparison of moving-boat measurements and current-meter measurements made on the Delaware river at Delaware Memorial Bridge. Again the results compare favourably, particularly near zero net flow.

Table 3 compares 14 moving-boat measurements made on the Mississippi river at St. Louis, Mo., with the rating-curve discharge over the same time period. During this measuring period there was a slight decrease in stream-flow, and the last column of the table represents the deviation from the rated discharge at the time of measurement. The average of the 14 moving-boat measurements is within 1.2 per cent of the mean rating-curve discharge.

Figure 4 shows the rating curve for the Mekong river near Vientiane, Laos. The entire curve is based upon moving-boat discharge measurements made over a period of two years. Most of the scatter that appears can be attributed to scour and fill of the gravel streambed.

Table 4 lists eleven moving-boat measurements made on the São Francisco river at Pirapora, Brazil. Flow conditions were steady over this period of time, and the average of the eleven measurements is within 1 per cent of the discharge determined from the stage-discharge rating curve.

Table 5 lists the results of four moving-boat measurements made on the Amazon river at Obidos, Brazil. Because only a few previous conventional measurements had been made at this site, the rating curve is not well defined and thus no comparative deviations from it were computed. Figure 5 is a plot of all previous measurements at Obidos, including the average of the four moving-boat runs which is denoted by the triangular symbol.

CONCLUSIONS

Results indicate that the moving-boat technique is a reliable and rapid method of measuring discharge on large streams. The rapidity with which a measurement can be made makes this method a valuable tool, especially in situations where flow conditions are varying. It is also very useful in making measurements where no permanent structures exist. Discharge measurements obtained by the moving-boat techniques agree within 5 per cent with measurements made by conventional means.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Oltman, Roy E. 1968. *Reconnaissance investigations of the discharge and water quality of the Amazon River.* (U.S. Geological Survey Cir. 552.) 16 p.

Smoot, G.F.; Novak, C.E. 1969. Measurement of discharge by the moving-boat method. *U.S. Geological Survey Techniques, Water Resources Inv.*, book 3, chap. A11, 22 p.

Sousa, Stelio. 1968. *Medicao de descharga e seus problemas tecnicos no maior rio du mundo.* Rio de Janerio, Divisao De Aguas, 42 p.

DISCUSSION

Bielenski: 1. During one measurement, was the current meter fixed at one depth only, or at several depths?

2. Have measurements been carried out during a full tide or only during a small interval?

3. Did you measure tidal discharges?

Smoot: 1. The current meter was fixed approximately 1 m below the water surface, regardless of the stream depth.

2. We have made tidal measurements over long periods of time. We cannot discriminate the discharge of a tidal estuary by comparing the total flow measured for the ebb and tide with the inflow. In the example I gave on the Hudson river at Poughkeepsie we had maximum discharges of 240,000 cfs (7,000 m^3/s) plus and minus but we had only a total discharge of about 10,000 cfs (280 m^3/s) flowing in the stream itself.

Garg: What is the highest discharge measured by the moving boat method?

Smoot: In the Amazon river it is 87,000 m^3/sec.

Garg: What was the coefficient between the mean velocity and the sub-surface velocity of that discharge?

Smoot: The coefficient was 0.94. It was not influenced by the discharge but rather by the depth. The Amazon river is approximately 200 feet (60 m) deep and the vertical velocity profile is very straight vertically.

Tywoniuk: I am surprised at the velocity of traversing the stream which you use during your moving-boat measurements. The speed with which you measure obviously contradicts the accepted fact that 40 seconds' time are required for each point velocity measurement. Two points per vertical and 20 verticals in a cross-section result in a total measurement time of 1,600 seconds. However, your results are very good. How can this be explained? It is that the pulsations in flow are averaged out as the boat moves?

Smoot: With this method we would use 30 to 50 sampling points. Any one sampling point may give you 2 higher or 2 lower velocities due to these fluctuations or even due to some deviation of travelling across the pair. But, if you remain fairly well on pair and sample at so many points, then you average out the fluctuations.

Faber: Do you keep the boat speed constant or does it vary? What is the effect of boat speed on the final figure?

Smoot: The path is maintained as much as possibly by steering rather than changing the boat speed. Obviously, since we use both the sine and cosine the optimum angle would be approximately 45 degrees. However, you cannot travel and control the boat at a velocity much less than $2\frac{1}{2}$ to 3 feet/sec (75 to 90 cm/s). But if the stream velocity is lower, the results will not be so good, according to this theory. In spite of this, all results were good. Even at the zero discharges the results were excellent.

Faber: But do you try to keep the boat speed constant or do you measure the boat speed?

Smoot: We do not measure the boat speed. We make an effort however not to fluctuate the boat speed too much.

Van Eyden: Do you have any special experience with your moving-boat discharge measurement in estuaries with density currents, due to salt penetrations from the sea ? In these estuaries, there are special velocity distributions in the vertical, i.e. different current directions, etc. during high and low water slack. How do you determine your average cross-section coefficient to be applied to the total discharge in these cases?

Smoot: I admit that it is not desirable to have salt penetration and velocities in both directions. Since we have no better technique of measuring tidal reaches, we usually locate one or more cruises making vertical traverses during the whole period of measurement. Then those coefficients are related to the results; the traverses are made during the normal measurement.

Ven Eyden: But those coefficients will vary momentaneously.

Smoot: That is absolutely true. The coefficient is determined for one discharge measurement only. Another has to be determined for the next measurement.

Oebius: During your experiments, did you find some differences in the movements of the boat when using it in deep and in shallow water, owing to the effect of shallow water on moving bodies?

Smoot: We have not recommended the use of the moving-boat method in water of less than 10 feet (3 m) primarily because of the vane. We have tested the device in water ranging from 10 to 210 feet (3 to 63 m) deep.

Rodier: Nous pensons que la méthode décrite peut rendre de très grands services pour l'étude des très grands fleuves comme le Congo ou l'Amazone et surtout des fleuves à marée ; il existe une méthode intermédiaire entre la méthode classique et la méthode du "moving boat" : c'est la méthode de jaugeage par intégration. Dans cette méthode, le bateau s'immobilise sur chaque verticale, mais le moulinet descend de façon continue et à une vitesse uniforme à partir de la surface jusqu'au fond et c'est la vitesse moyenne qui est mesurée. Nous avons employé cette méthode au Gabon sur l'Oroué, pour des débits variant entre 2 000 m^3/s et 8 000 m^3/s. Mais certaines précautions sont à prendre, sinon des erreurs systématiques de 5 à 10 % sont possibles. Nous pensons également que la méthode qui vient d'être décrite demande beaucoup de soins ; avant de commencer les mesures sur un fleuve, il est en particulier nécessaire de vérifier le type de distribution.

Wiesner: I should like to add some observations on the magnitude of errors of measurements with current meters. The error is contained in the result. Its magnitude, the difference between the measured value and the real one, cannot normally be determined. It is however possible to determine this difference by applying larger series of measurements.

With regard to the measurements of the Spree river, these series were applied. An attempt was made to derive a methodological rule by applying the continuity equation. Several cross-sections were established with equal distances. They were located below a weir in order to guarantee a constant discharge and to exclude systematic errors of measurement which may arise from differences in the profile. The measurements were carried out strictly following the official rule.

We applied the six-point method by measuring the velocity in 10 verticals at a depth of 0.8, 0.6, 0.4, 0.2 and directly under the surface as well as above the bottom. One measurement lasted 40 sec. On the assumption that the discharge is constant ($dQ/dt = 0$), the n repetition can be regarded as a series. The real discharge was regarded as the arithmetic mean of the measurement. The error therefore is the difference of a single measurement and the mean.

The relative errors follow a straight line on the probability paper and thus correspond to the Gaussian distribution and therefore are random errors. Comparing the several methods, we found the following coefficients:

six point method: 3,6 per cent
two-point method (0.2 and 0.8): 4.3 per cent
KREBS method: 4.7 per cent.

The probability of an error would then be:

six-point method: 10 per cent probability = ± 0.5 per cent error
six-point method: 20 per cent probability = ± 1.0 per cent error
six-point method: 50 per cent probability = ± 2.4 per cent error
six-point method: 80 per cent probability = ± 4.6 per cent error
two-point method: add one per cent.

Problems in the design of measuring structures

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Abstract. Head-loss, operating costs of measurement, tailwater submergence and the mean error of water volumes determined on the basis of observed rates of flow are the major aspects required to be considered in the design of structures for continuous flow measurement.

With the effect of submergence known and the original rating curve of the channel determined, the free-flow structure causing the smallest head-loss can be designed using the semi-graphical method described. Submergence causes an error, the magnitude of which can also be estimated from the duration curves. The method is illustrated by an example.

PROBLEMES DANS LE TRACE DES RESEAUX DE MESURES

Résumé. La perte de charge, les frais d'exploitation, l'influence du niveau d'aval et l'erreur moyenne sur le volume d'eau calculé sur la base des débits mesurés sont des points dont on doit tenir compte dans l'étude des ouvrages servant à la mesure continue des débits.

Connaissant l'effet du niveau d'aval et la courbe de débit naturelle du canal, on peut déterminer, à l'aide de la méthode semi-graphique décrite, les dimensions du jaugeur à écoulement libre accusant la perte de charge minimale.

L'erreur engendrée par la submersion peut être également évaluée sur la base des courbes des débits classés. La méthode est illustrée par un exemple.

PROBLEMAS EN EL PROYECTO DE ESTRUCTURAS DE MEDICIÓN

Resumen. Cuando se proyectan estructuras para la medición de caudales continuos, los principales aspectos que deben ser tomados en consideración son la pérdida de carga, los costos de funcionamiento, la formación de curvas de remanso y el error medio de los volúmenes de agua determinados en función de los valores de flujo observados.

Una vez conocido el efecto de la sumergencia y determinada la curva original de calibración del canal, se puede dimensionar la estructura de flujo libre que origina la menor pérdida de carga con el método semigráfico que se describe. La sumergencia causa un error cuya magnitud se puede determinar a partir de las curvas de duración. El método viene ilustrado con un ejemplo.

ПРОБЛЕМЫ, ВОЗНИКАЮЩИЕ ПРИ ПРОЕКТИРОВАНИИ ИЗМЕРИТЕЛЬНЫХ СООРУЖЕНИЙ

Резюме. Потеря напора, стоимости измерений, подтопления со стороны нижнего бьефа и средние ошибки объемов воды, определенные на основе измеренных величин расхода, являются важными аспектами, которые следует учитывать при проектировании сооружений для непрерыного измерения расхода.

Если известен эффект подтопления и установлена наyальная кривая расходов русла с помощью описанного полу-графического метода, могут быть определены размеры незатопленного сооружения, вызывающего наименьшие потери напора. Подтопление вызывает ошибку, величина которой может быть также оценена по кривым продолжительности. Метод иллюстрируется примерами.

The objective of continuous flow measurement is to record rates of flow, which vary within wide limits, especially on natural watercourses, and to compute total flow volumes together with various characteristic quantities derived therefrom. Difficulties may arise in the design of flumes and weirs intended for continuous flow measurement. For this reason it may be necessary to consider a compromise in accuracy requirements over a certain range of flow rates, depending on the particular objective of measurement. When dealing with problems of this nature, allowance must be made in the design also for the flow duration curve for the watercourse.

The basic problem in the design of measuring structures is to create by means of the structure a measuring head, from which the rate of flow can be computed with the desired accuracy. The main (wetted) dimensions of the structure are defined in this way. Measurement should, in general, be unaffected by tailwater stages, that is to say free flow conditions through the structure should be maintained. Substantially, the latter requirement depends also on the accuracy desired and the differential head created by the structure which increases as the requirements for higher accuracy and free-flow conditions become more stringent. Local conditions frequently prevent the designer from adhering to high standards of accuracy and to rely on the duration curve of the watercourse instead.

Selection of the free-flow measuring structure causing the smallest average annual head-loss

The condition of free-flow is that the tailwater stage must not exceed a limit which is characteristic of the structure. In the simplest case, this limiting stage, referred to as the limit of submergence, depends exclusively on the rate of flow (i.e. on the headwater stage) and at the limit the ratio of tailwater and headwater stages is constant:

$$\frac{h_{t,\,\mathrm{lim}}}{h_h} = d = \text{constant.}$$

In more complex cases d is also variable and then

$$h_{t,\,\mathrm{lim}} = f(Q, d) = F(f/Q),\ d.$$

Quite obviously the smallest head-loss is caused by a particular structure, if the differential head between tailwater and headwater is lowest in the range of free flow, i.e. when the tailwater stage is at the limit of submergence.

This consideration leads to the approach outlined in Fig. 1. Dividing the ordinates h_t of the rating curve for the tailwater channel by the ratio d, a fictitious transformed rating curve is obtained for the headwater channel, which defines the headwater stage pertaining to the submergence limit. The upstream rating curve of the measuring structure must always remain above this curve, or it may coincide with it.

If a suitable structure without a raised sill can be found, then the study may proceed without further difficulty. If, however, due to the shape of the rating curve of the structure, the desired position of the curve can be attained by raising the sill only, then the submergence limit must be checked for the sill elevation. This is done by multiplying the ordinates of the rating curve of the structure by the submergence limit ratio pertaining to the raised sill. The structure will always operate under

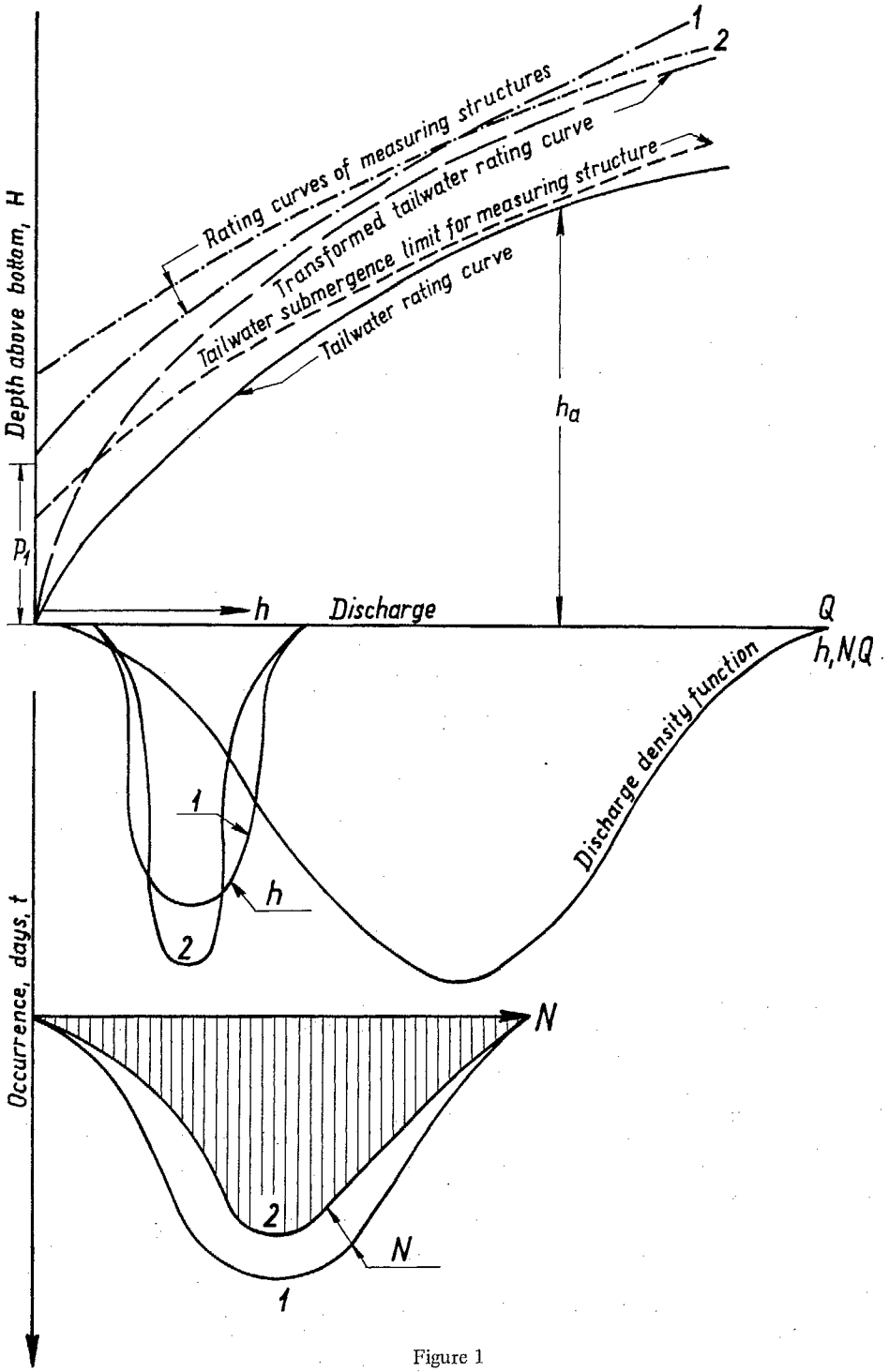

Depth above bottom, H
Rating curves of measuring structures
1
2
Transformed tailwater rating curve
Tailwater submergence limit for measuring structure
Tailwater rating curve
h_a
p_1
h
Discharge
Q
h,N,Q
Discharge density function
1
h
2
Occurrence, days, t
N
2
N
1

Figure 1

free-flow conditions if the curve thus obtained is situated above the rating curve of the natural channel.

A number of structures meeting this condition can be designed in terms of the type and dimensions (sill height p and throat width L) of the structure affecting the backwater. The problem is to select one which will cause the smallest average annual head loss. This can be accomplished on the basis of the frequency curves of flow, head-loss and power loss derived from the first two.

Once the frequency curve of flow rates is available, that of head-loss is found by plotting the differences between the ordinates of the rating curves for the tailwater channel and the structure at a particular flow rate in the horizontal corresponding to that rate of flow. It will be recalled that at the head-loss h and flow Q the resulting loss of power is

$$N = \gamma \cdot Q \cdot H = 1\,000\ Qh \quad [\text{mkg/s}] = 9.8\ Qh \quad [\text{kW}].$$

The frequency curve of power losses is thus obtained by multiplying the corresponding Q and h values and entering these on the abscissa. It will be perceived that the annual loss of power is proportional to the area under the frequency curve of power losses, i.e.

$$E = \int_0^T N\ \mathrm{d}t = \int_0^T 9.8\ Qh\ \mathrm{d}t.$$

From among the energy losses determined thus for different measuring structures, the one yielding the lowest value will indicate the optimum type of structure, or the optimum dimensions as far as energy losses are concerned.

Determination of the operating costs of measurement due to energy losses

At any measuring site established on irrigation or drainage canals an energy loss is thus created, which is reflected in the operating costs due either to higher costs of pumping, or to the reduction of the area served. The annual loss of energy is multiplied by the specific cost of energy to obtain—at efficiency—the annual cost K of operation:

$$K = \frac{9.8a}{\eta} \cdot \int_0^T Qh\ \mathrm{d}t.$$

In general it may be assumed that under prevailing conditions the specific cost of operation for lifting 1 m^3 of water to a height of 1 m is on average constant, k, and thus for a particular site the value $k = 9.8a/\eta$ may be entered into the computations:

$$K = k \cdot \int_0^T Q \cdot h\ \mathrm{d}t.$$

This relationship is illustrated in Fig. 2.

In some instances head losses can be expressed in terms of flow in the form $h = f(Q)$ and thus operating costs can be related directly to the rate of flow. In the simplest case of flow under pressure

$$h = bQ^n,$$

where b and n are constants, with the value 1/2. In this case,

$$K = bk \cdot \int_0^T Q^{1+n} \cdot \mathrm{d}t$$

is the function to be used in preference, so that the integral frequency curve of values Q^{1+n} must be produced for characterizing energy losses. Related to the bottom sill P, neglecting velocity heads:

$$h = P + H - h_t.$$

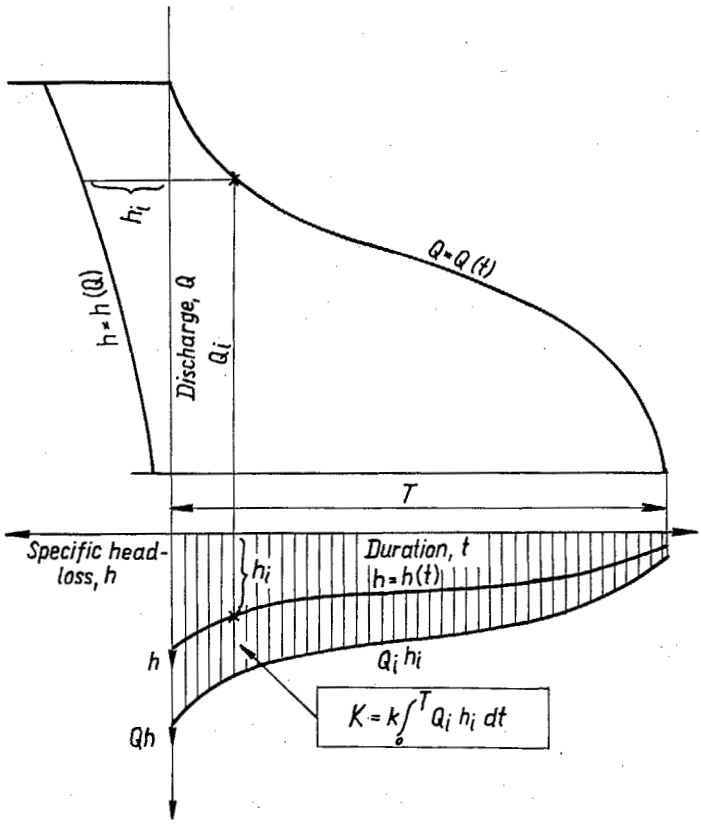


Figure 2

The value from $A = MLH^r$ may be substituted in the formula for H and that from $Q = Ah_t^m$ for h_t. The formula then becomes

$$h = P + \left(\frac{Q}{ML}\right)^{1/r} - \left(\frac{Q}{A}\right)^{1/m},$$

where M is the discharge coefficient, L a characteristic dimension, while r, m and A are constants.

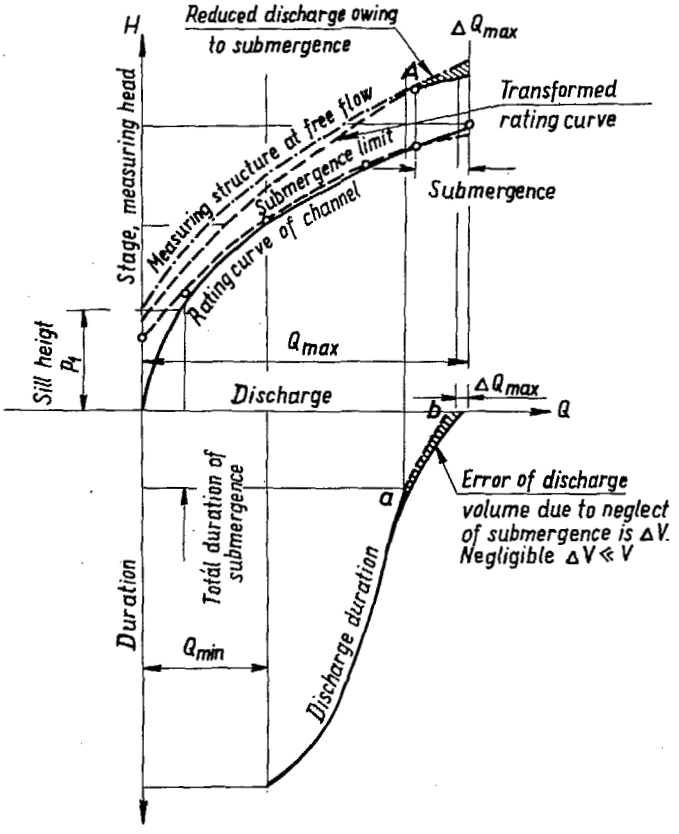


Figure 3

The above relationship is used to find the energy loss directly from the flow rate Q, or it can be read as in the case outlined in connexion with the upper field of Fig. 1.

The error introduced by neglecting the reduction of flow due to submergence

Where the influence of the tailwater cannot be eliminated completely and submergence occurs beyond certain rates of flow, the error introduced must either be allowed for in the determination of flow volumes, or its magnitude must be estimated.

The effect of the error on the annual flow volume due to submergence can also be estimated with the help of the duration curve (Fig. 3). Where the transformed rating curve of the tailwater channel intersects the rating curve of the structure pertaining to free-flow conditions, i.e., beyond the point A, flow is reduced by submergence by the value ΔQ compared with free-flow conditions. (The value ΔQ can be found from the family of Lamoen curves for the structure.) The reduction ΔQ in flow is

represented on the flow duration curve by the section *a-b*, or the change in volume ΔV is proportional to the area enclosed by the original and new curves. The relative error introduced by neglecting the influence of submergence is thus expressed by the ratio $\Delta V/V$. A deviation of a few per cent is in general considered permissible and the involved computation of submergence is regarded as superfluous (at least as far as annual runoff volume is concerned).

The mean error of volume estimation

Annual volumes are estimated mostly from daily mean flow rates but sometimes from the area under the flow hydrograph using the formula

$$V = \Sigma Q_i \cdot \Delta t.$$

The volume thus determined involves a mean error σ_v, the magnitude of which is according to the relationships of mathematical statistics

$$\sigma_v = \Delta t \cdot \sqrt{\Sigma \sigma_Q^2};$$

where Δt is the length of intervals and σ_Q the mean error of flow measurement. When dividing the year into n intervals Δt, then

$$\sigma_v \quad \Delta t \cdot \sqrt{n} \cdot \sigma_Q$$

It is interesting to note that the relative mean error of volume estimation is substantially smaller than the relative mean error of flow measurement:

$$\frac{\sigma_v}{V} = \frac{\Delta t \cdot \sqrt{n} \cdot \sigma_Q}{\Delta t \cdot n \cdot Q} = \frac{1}{\sqrt{n}} \frac{\sigma Q}{Q}$$

or it may be smaller by $1/\sqrt{n}$.

The evaluation of discharge measurements in streams with changing flow conditions

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Abstract. So far, discharge can only be measured discontinuously. The discharge hydrograph for the time interval between two measurements is obtained from the stage hydrograph by means of the stage/discharge curve.

Widely scattered values obtained for the stage/discharge relationship may be caused not only by weed conditions but also by moving sand, bank erosion or backwater. The values obtained at the Bienenbüttel decade gauge on the Ilmenau river ($F_N = 1{,}457$ sq. km) are taken as an example. With the aid of the improved Gils method, an obstruction factor η is derived from the fluctuations of the measured values which is representative of the water's discharge behaviour at the time of measurement. The η-hydrograph describes the changes in discharge as caused by weed, changes in the river bottom and backwater during a discharge period.

EVALUATION DES MESURES DE DEBITS EN RIVIERE AVEC MODIFICATION DES CONDITIONS D'ECOULEMENT

Résumé. Actuellement, le débit ne peut être mesuré que d'une manière discontinue. C'est à partir de la courbe du niveau d'eau qu'on exploite les résultats des mesures du débit d'eau.

Dans certaines conditions spéciales qu'on trouve dans les pays plats — en Basse-Saxe, plus de 90 % des cas — la courbe du débit d'eau ne peut être obtenue qu'après un traitement particulier des valeurs mesurées, étant donné que la croissance des plantes, les sables mouvants, l'érosion des berges et le reflux produisent une grande dispersion de ces valeurs.

Les valeurs obtenues par la jauge de Bienenbüttel, sur l'Ilmenau (région des précipitations $F_N = 1\ 457\ \text{km}^2$) sont citées comme exemple de classement d'interprétation des valeurs mesurées dispersées. Les méthodes employées jusqu'ici pour classer les valeurs sont expliquées en quelques phrases. Une adaptation de la méthode de Gils permet de déterminer à partir des variations des valeurs mesurées le coefficient de ralentissement η du débit d'eau, qui est caractéristique de la nature du débit au moment de la mesure. L'hydrogramme de η décrit les variations du débit dues aux plantes, aux changements de fond, au reflux.

EVALUACIÓN DE LAS MEDICIONES DE CAUDAL EN CURSOS DE AGUA CON CONDITIONES DE FLUJO CAMBIANTES

Resumen. En estas condiciones, el caudal sólo se puede medir en forma discontinua. El hidrograma de caudal para el intervalo comprendido entre dos mediciones se obtiene por medio de la curva de caudal, a partir del hidrograma de nivel.

El autor explica la forma en que se puede determinar el nivel en el caso de resultados fuertemente fluctuantes, que pueden deberse a condiciones no sólo de crecimiento de vegetación perjudicial, sino también al desplazamiento de arenas, a la erosión de las márgenes o a efecto de remanso. Los valores de medidas obtenidos en la estación de medición del Decenio en "Bienenbüttel" en el río Ilemnau ($1\ 457\ \text{km}^2$) se toman como ejemplo. Con ayuda del método "Gils" mejorado, se obtiene un coeficiente de obstrucción η de las fluctuaciones de los valores de medición, que es representativo de la conducta del agua en el momento de la medición. El hidrograma η describe los cambios en el caudal debidos a la vegetación, cambios en el fondo del río, remanso, etc., durante un periode de medida.

ОЦЕНКА ИЗМЕРЕНИЙ РАСХОДОВ НА РЕКАХ С МЕНАЮЩИМИСЯ УСЛОВИЯМИ ТЕЧЕНИЯ

Резюме. До сих пор расходы могут измеряться только дискретно (с перерывами). Гидрограф за интервал времени между двумя измерениями получают, пользуясь кривой расходов по графику колебания уровней.

Автор поясняет, как можно определить расход в случае сильно меняющихся результатов, которые могут быть вызваны не только условиями зарастаемости русла, но также движущимся песком, эрозией берегов или подпором. Измеренные значения, полученные на реке Ильменау ($F_N = 1457$ км^2), рассмотрены в качестве примера. С помощью улучшенного метода «Гилса» получен коэффициент препятствия η флюктуаций измеренных значений, который характеризует режим расхода воды во время измерения. Гидрограф коэффициента η описывает изменения в расходе, вызванные зарастанием, изменениями в речном дне, подпором и т. д. в течение периода между измеренными расходами.

INTRODUCTION

The Bienenbüttel gauge on the Ilmenau river, which is a left-hand tributary of the Elbe river, coming under the Water Management Board of the Lower Saxonian city of Lüneburg, is one of the measuring stations operated within the framework of the International Hydrological Decade. Located about 45 km upstream from the mouth of the Ilmenau river, it controls 1,457 sq. km of the catchment's total area of 2,884 sq. km. As a discharge measuring station, it is characteristic of the entire Lüneburg Heath.

It has been impossible to obtain a general stage-discharge relationship because the measured values are too widely scattered. The present paper uses the Bienenbüttel decade station as an example to show how the causes for the scatter of values can be determined and how a distinct relationship between stage and discharge can be re-established. It is even possible to deduce information on the discharge character of the river at the time of measurement.

Since 1887, stage has been observed at several gauges. The Bienenbüttel gauge (first-order gauge) has been in operation as a recording gauge since 1 November 1953. During the summer months, discharge is retarded primarily by weed growth. The relationship between stage and discharge may also be influenced by sand transportation and bank erosion.

EVALUATION OF WIDELY SCATTERED RESULTS

General

Within a few years, the introduction of the two-point measuring method resulted in the accumulation of a large amount of data. It was then found that for many gauges a stage-discharge curve could not be developed graphically, on account of the wide scatter of measured values. Doubts about the reliability of numerous previously established discharge curves were confirmed. The prerequisite conditions for such gravity discharge curves were lacking in these cases because the discharge of the streams concerned is more or less retarded.

The impossibility of establishing the discharge curve by conventional methods stimulated the search for other ways of evaluation which would justify the continued

operation of the gauging station. Since the early thirties, reports have repeatedly been published (*inter alia* by Pantle 1952, Gils 1962 and 1966, Schenk 1965) on the difficulty of obtaining reliable discharge values and, consequently, unambiguous discharge curves for streams with retarded flow, and on the possibilities of evaluation by way of determining the weed factor, retardation factor, or impoundage. Where stage fluctuates, as is normally the case, Gils' η-method has been found most reliable and exact. The evaluation of more than 15,000 measurements showed that η is not only a "weed factor" but indicates the degree of discharge retardation of any kind occurring within the gauge area. The η-method is a very efficient procedure for covering, generally, any change in flow conditions (sand movement, profile modifications, impoundage, weed growth and run-off changes during a discharge period).

Evaluation according to Gils' improved method

The removal of weeds and similar obstacles in preparation for the discharge measurement is required only in the immediate vicinity of the measuring point so as to permit the unimpeded operation of the current meter. The measured values (W and Q) obtained, as a matter of routine, once a month over years are marked in different ways according to the time periods

January-April, May, June, November, December
July-October

and are plotted as points in a system of co-ordinates (ordinate: water stage; abscissa: discharge). Seasonal influence (weed growth) on water stage under stable discharge conditions can thus be recognized. Winter values are the lowest. A scattering of these measuring values may indicate other causes of discharge retardation.

As a basis for the future evaluation of discharge measurements, the enveloping boundary lines should be drawn after the HHQ_0-value[1] has been calculated and the HHW_0-value[1] estimated. The determination of HHW_0, calls for experience and familiarity with local conditions.

HHW_0 and bottom elevation (generally equal to the maximum water depth when proceeding from mean water values) are the points of departure for the graphical representation of the boundary lines. The boundary lines should be smoothly adapted to the dispersion field of the measured points; they should not touch the measured values in the extreme locations but should leave on both sides of the dispersion field a margin of just one-tenth the width of the dispersion field. This leaves space for future extreme values, the occurrence of which cannot be ruled out. The lower boundary line is the curve of unobstructed discharge (Q_0), the upper boundary line the discharge curve at maximum discharge retardation (Q_z). The abscissa difference between both curves is the maximum discharge retardation ($Q_h = Q_0 - Q_z$).

For use during discharge measurements and for subsequent evaluation, a numerical table (formerly a co-ordinate chart for graphical representation) is developed for each gauging station ("Table of discharge boundary values"). In this table, the discharge values for Q_0 and Q_h are allocated to each water level.

$$\frac{Q_0 - Q}{Q_h} = \eta \quad = \text{discharge retardation factor}$$

Q = measured (calculated) discharge.

1. For explanation, see notation.

With the help of the numerical table and two measurements taken from one discharge period, the η-values can be determined. Provided that η does not change suddenly, values may be interpolated for the days between and the daily discharge Q can be calculated from the daily observed water levels, the corresponding η-values, the Q_0-value, and the Q_h-value. $Q = Q_0 - \eta \cdot Q_n$ (m^3/sec).

If HHW_0 should change, the upper branches must be adapted to the boundary lines, and the upper range of the numerical table (table of discharge boundary values) must be compiled anew. If, for example,in the case of watercourse improvement, the boundary lines change substantially, a new numerical table must be compiled. The η-values of the old table cannot be compared directly with those of the new table.

The η-hydrograph and its interpretation

In the course of a measuring period covering a number of years, diversified η-values are obtained which can be expressed graphically as the hydrograph of the discharge retardation factor. The η-hydrograph of the Bienenbüttel gauge for the period 1956-69 shows, seasonally as well as over the years, fluctuations in discharge retardation. A comparison of the mean values of the discharge retardation factor η, the total of radiation, and the air and water temperature for the said period (1956-69) reveals the interdependence of these measured values.

Thus, for example, the hydrograph of the mean discharge retardation η reaches its peak about 45 days later (i.e., early in September) than the hydrograph of the mean water temperature, and about 75 days later than the hydrograph of total radiation.

The extreme values of the boundary lines for the period 1956-64 have their maximum values between $\eta = 0.5$ at the end of March and $\eta = 1$ early in September, and their minimum values between $\eta = 0.3$ early in April and $\eta = 0.6$ early in September. They show that the range of seasonal fluctuations of η is comparatively large, without reaching zero. This is due to:

1. Weather influences (hot - cold, duration of sunshine);
2. Run-off conditions (distribution and intensity of precipitation);
3. River bottom changes;
4. Effects of impoundage upstream or downstream from the measuring point;
5. Miscellaneous effects (for instance, eutropification as a result of the excessive application of fertilizer to agricultural areas).

If standard relationships between η, total radiation, air and water temperature, are known on the basis of the run-off conditions, deductions as to the weather régime of the year concerned and/or changes in the river bed or banks, or as to the presence of effects resulting from impoundage may be made from the analysis of the deviation of the factor η from the mean value. In this case, the long-term record and evaluation of the discharge retardation factor η and other measuring values may permit a general assessment of the discharge character of the watercourse.

With due caution, it may even be possible to draw, for an earlier period, conclusions on discharge from a stage reading recorded at that time, provided the weather régime is known. It should, however, be remembered that other influences such as changes of the river bed, impoundage caused by bridges and so on must then be excluded.

As against these very general deductions (much material must still be collected to improve evaluation), the η-hydrograph of a period describes changes of discharge caused by weed growth, changes in the river bottom and impoundage directly. A

comparison of the η-value obtained on the day of the measurement with the preceding η-values and the mean η-hydrograph shows quickly whether or not the measured value is representative or whether particular influences must be taken into account. Such information can be derived only if one starts to calculate, immediately after the measurement, the component discharges by the two-point measuring method. After the result of the last velocity measurement has become available, the total discharge must be determined. Immediately thereafter, the η-value should be calculated from the numerical table. The new η-value is compared with the η-value of the preceding measurement. If the η-value deviates considerably from the expected value, one should check immediately to ascertain the correctness of the calculation and of the reference material (velocity chart, current meter data and the numerical table). If the possibility of a fault can be excluded, and if the deviation continues to be inexplicable, the entire discharge measurement should be repeated at once.

If the new η-value deviation corresponds to expectations, the measurement may be considered as representative of the discharge behaviour of the watercourse on this particular day.

Originally, η was considered merely as a weed accumulation factor. It is now thought that, depending on the slope of the watercourse, diverse influences may affect the relationship between water stage and discharge and, consequently, also the η-factor, several kilometres down stream from the measuring point, and in exceptional cases also close upstream. Within this critical section, the watercourse must be thoroughly examined for changes that may have occurred since the last measurement was taken.

Critical analysis of a heavily modified discharge retardation factor

The critical analysis of the heavily modified η-value proceeds from the assumption that in the case of weed growth η increases steadily from April to October, and drops again from October to April. Weed removal times (sudden changes of η) must be taken into account. If an η-value has been determined that deviates more than the prevailing weed growth would seem to explain, the soudings of the discharge cross-section should be checked before the search for other causes is taken up. These causes may be as follows:

Measurements taken at discharge rates not substantially above MQ:

A. η has increased considerably. Possible causes:

(a) Constriction of the discharge profile on account of sand, sludge or gravel deposition, bank erosion, floating obstacles (e.g. a sunken boat), ice jam, or construction work at or in the watercourse (bridge or similar construction);

(b) Premature run-off of a flood wave of a tributary flowing into the watercourse at a downstream location within the critical range or a backwater effect in the stream;

(c) Changes at dams and weirs (impoundage).

B. η has dropped considerably. Possible causes:

(a) Enlargement of the discharge profile on account of sand, sludge or gravel shifting, breaking-up of an ice cover, weed removal, bottom clearance, intensified care of embankments, construction work at or in the watercourse (bed improvement);

(b) Quick run-off of a flood wave (lowering of the water level) of a tributary flowing into the watercourse at a downstream location within the critical range, or of the main stream, leading to a relative drop in water stage;
(c) Changes at dams and weirs.

Measurements taken at high water:

In the course of the passing of a flood wave, η is subject to changes in consequence of the increase in slope resulting from the rise of the flood wave, of the slope equalization at the flood wave peak, the reduction in slope during the run-out of the wave, and the weed being pressed down by the higher discharge. With increasing discharge, η becomes smaller than normal. More water is discharged within a comparable depth. Q approaches the value Q_0. When the water level drops, η increases beyond the original value, due to the reduction in slope. However η will not increase too much since the weeds will not be fully erect because the discharge is still higher than normal, and this will have a reducing influence on η. After the flood wave has passed, η will fluctuate back to the normal value.

For discharge measurements during the passing of a flood, the changes in water stage should, therefore, be recorded at close time intervals (the latter depending on the steepness of water level rise or fall).

If, during the passage of a flood wave, η-values are found which contradict these considerations, for instance, if η increases in spite of the rising water level, an additional discharge retardation element may be assumed to be involved, for example, backwater caused by a flood-carrying tributary flowing into the watercourse at a downstream location. It might however, also be possible that the bed downstream from the measuring point was overloaded with gravel during a previous flood and cleared later on, although higher discharges are still being obstructed.

Correspondingly, a reduction in η at the time of a falling water level may indicate the removal of an existing discharge obstacle by the flood (e.g. a cavity in the bed downstream from the gauging station).

If an abnormal η-value was obtained during a flood, measurements should be repeated after the flood wave has passed. It will then be possible to see whether or not this changed η-value still applies. In this connexion, attention should be drawn to a remarkable observation which follows from the hydraulic calculation in connexion with the graphical evaluation and comparison of the η-values and K-values. It would seem that under certain discharge conditions covering, in particular, the fluctuating range of the magnitude of the discharge, the tendency toward weed growth, the geomorphology of the valley bottom and the type of soil, there exists for a section of a watercourse a characteristic η-value which can always be reproduced mathematically at a certain season, even after the bottom has been cleared or the bed corrected (without artificial revetment of banks).

The mean K-values of the hydraulic calculation for MQ and S_0MHQ at the gauging point in the Ilmenau river are bound to a comparatively narrow η-value dispersion range. The K-value depends on the mean discharge depth (water stage). At a given water level, a lower K-value means a higher η-value, and vice versa. Since close relationships exist between η and K, it follows that, for the dimensioning of a water bed, a certain K-value must necessarily be used again and again as a characteristic of the section concerned. Thus, it is possible to consider the η-value also in this respect as being representative of the discharge character of a watercourse at the gauging point at the time of measurement.

CONCLUSIONS

The η-method has been used and approved for years in the evaluation of discharge measurements in the waters of the Lower Saxonian plain and other areas. The basic principles of the evaluation methods are described so that the method may be applied elsewhere under comparable conditions. It has been found that this η-method not only permits the consideration of weed growth in the evaluation of stage and discharge measurements, but also – through careful analysis of the η-value – it becomes possible to draw conclusions from the η-hydrograph as to changes in the stage-discharge relation which are due to changes in bottom conditions, impounding effects, bank erosion or other events including the erection of hydraulic structures which may not be known. Measuring errors caused by these events can be avoided. Thus, the discharge retardation factor η is representative of the discharge character of the watercourse at the time of the measurement, provided that the measuring and evaluating methods have been applied properly.

NOTATION

HHQ_0	Maximum unobstructed discharge ever measured.
HHW_0	Maximum unobstructed water stage ever measured.
K	Coefficient in the discharge formula, expressing the roughness of the river bed.
MQ	Mean discharge.
Q	Daily discharge.
Q_0	Unobstructed discharge.
Q_h	Difference between unobstructed and extremely retarded discharge at one and the same water level.
Q_z	Extremely retarded discharge.
W	Water stage.
η	Discharge retardation factor.

Figures 1 to 7: pages 176 to 180.

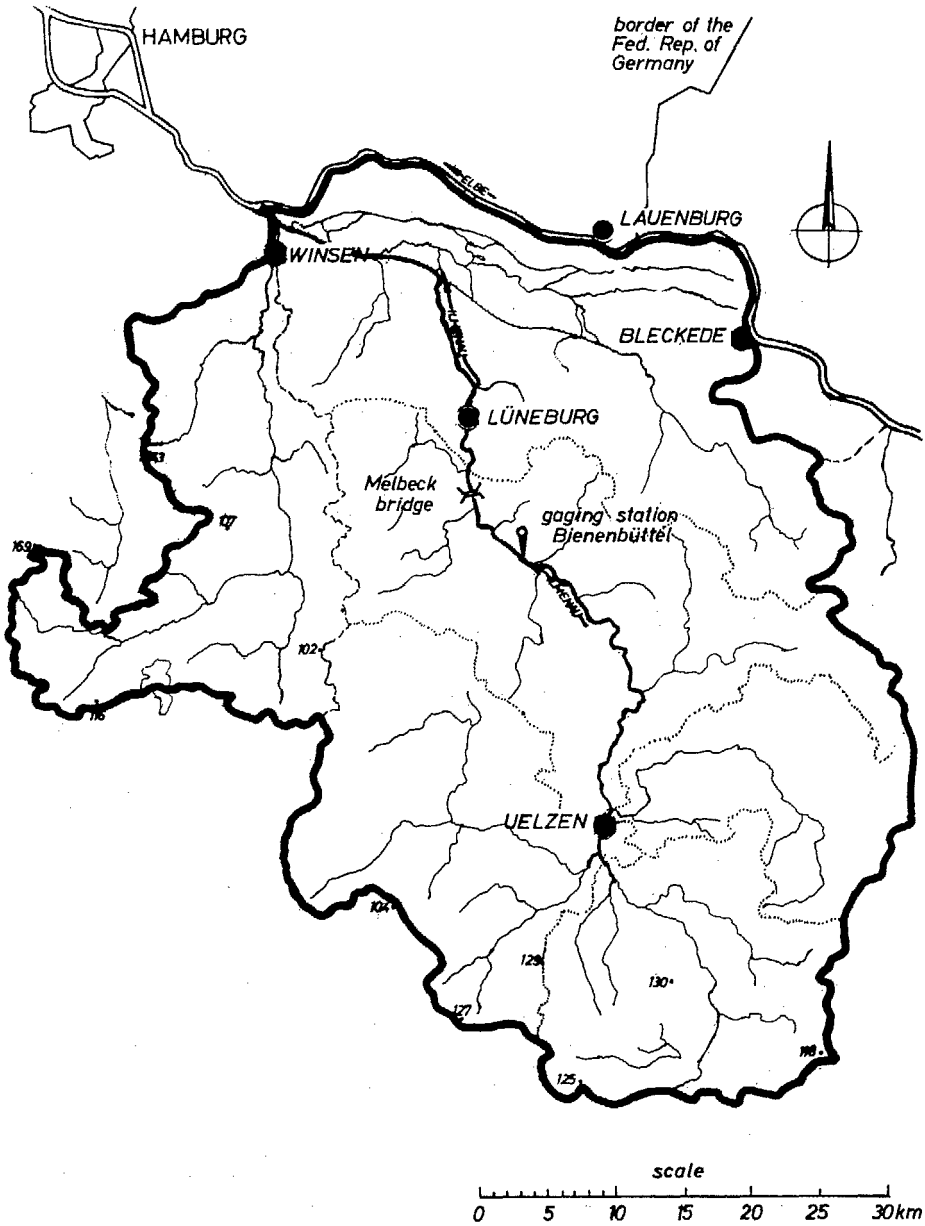


Figure 1. *Catchment area of the Ilmenau river*

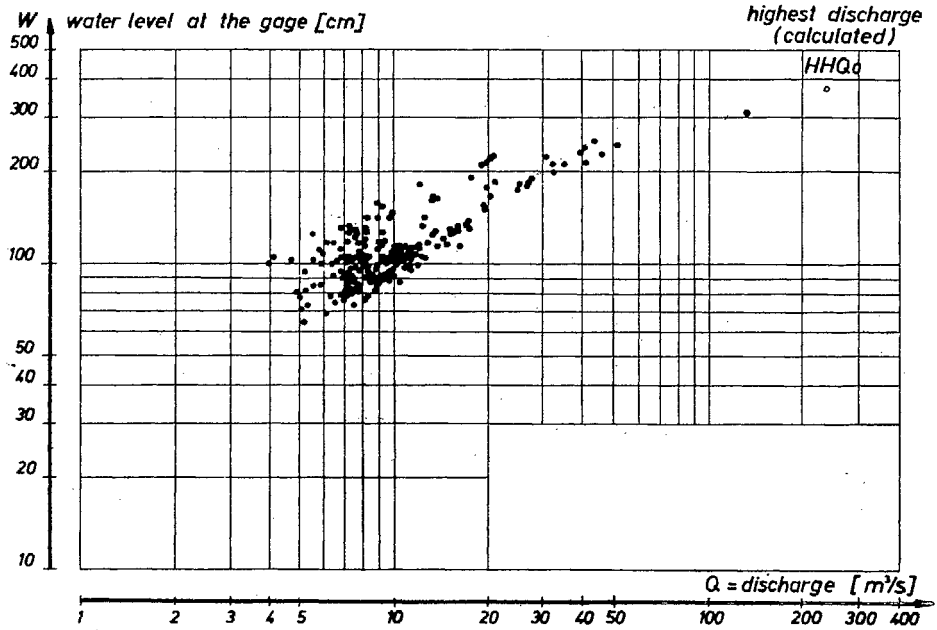


Figure 2. *Observed discharge values at Bienenbüttel gauging station, unsorted*

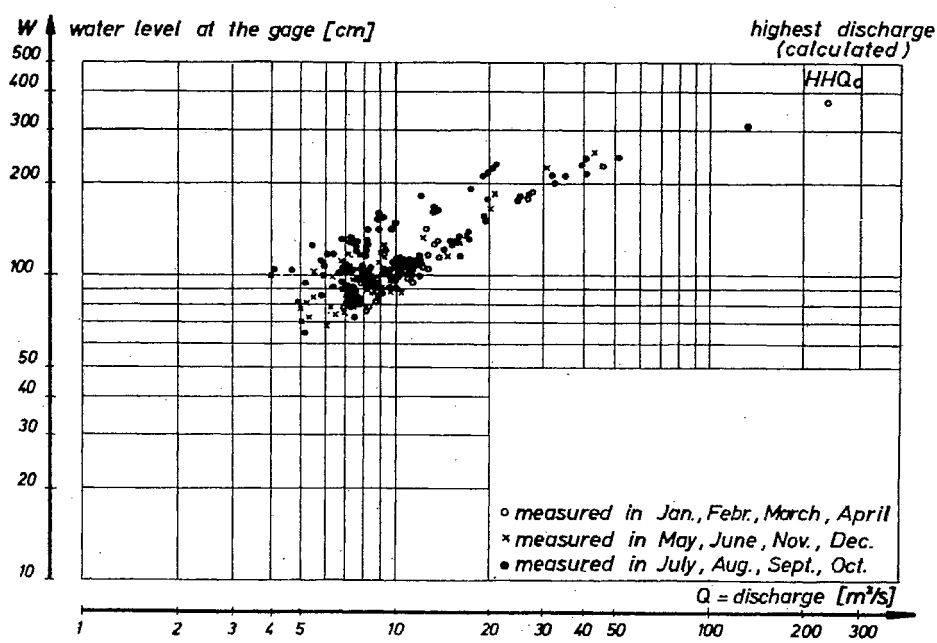


Figure 3. *Observed discharge values at Bienenbüttel gauging station, seasonally sorted*

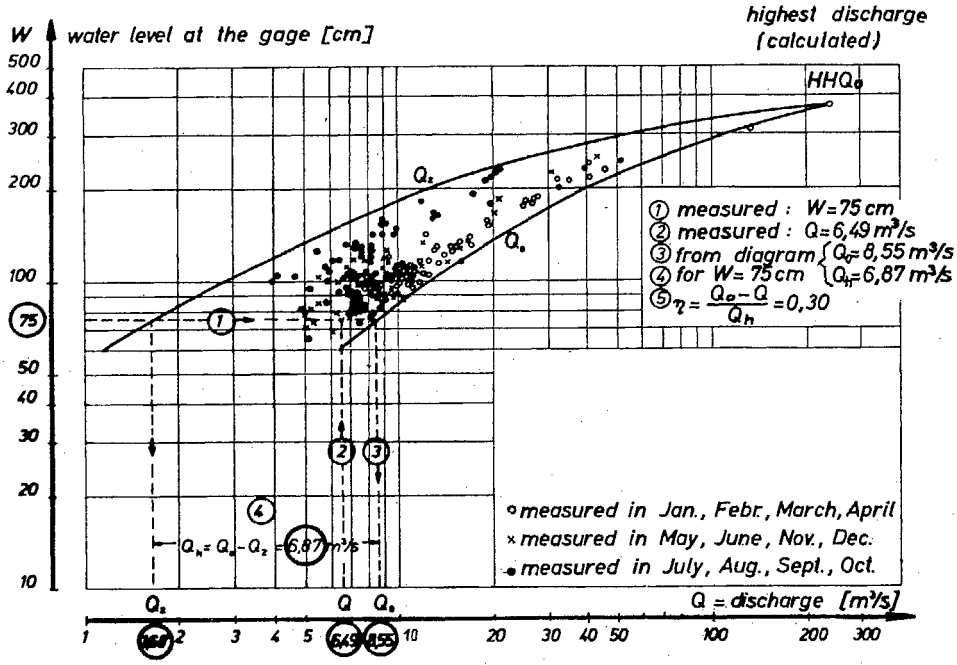


Figure 4. *Observed discharge values at Bienenbüttel gauging station, seasonally sorted with limiting lines*

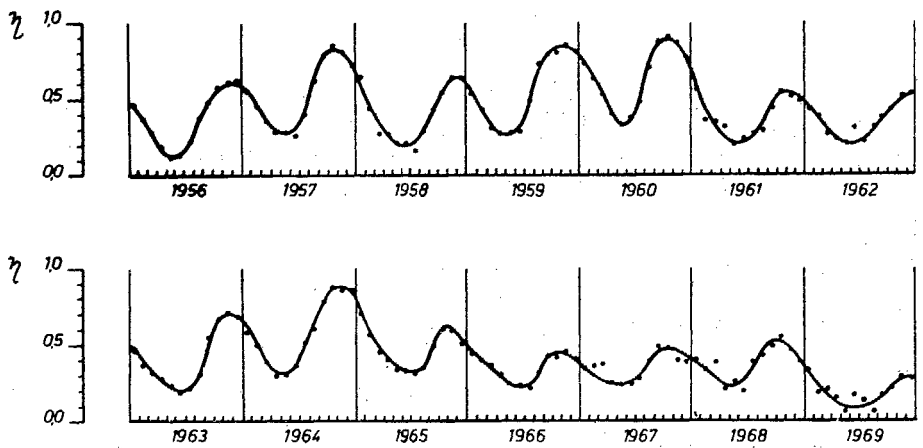


Figure 5. *η-hydrograph, gauging station Bienenbüttel (water years 1956-69)*

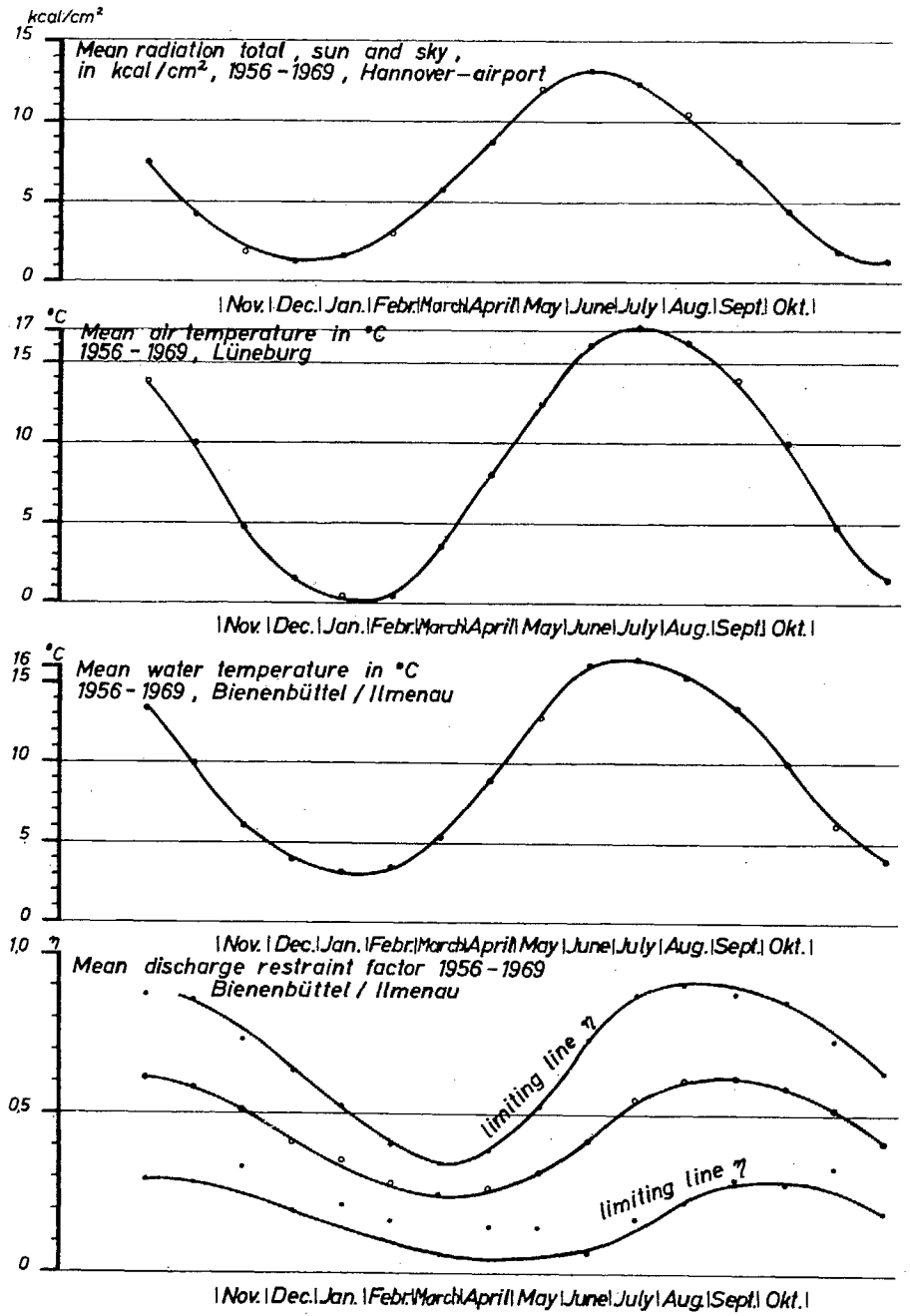


Figure 6. *Total radiation, air temperature, water temperature, discharge restraint factor, hydrograph of the monthly mean values (water years 1956-69)*

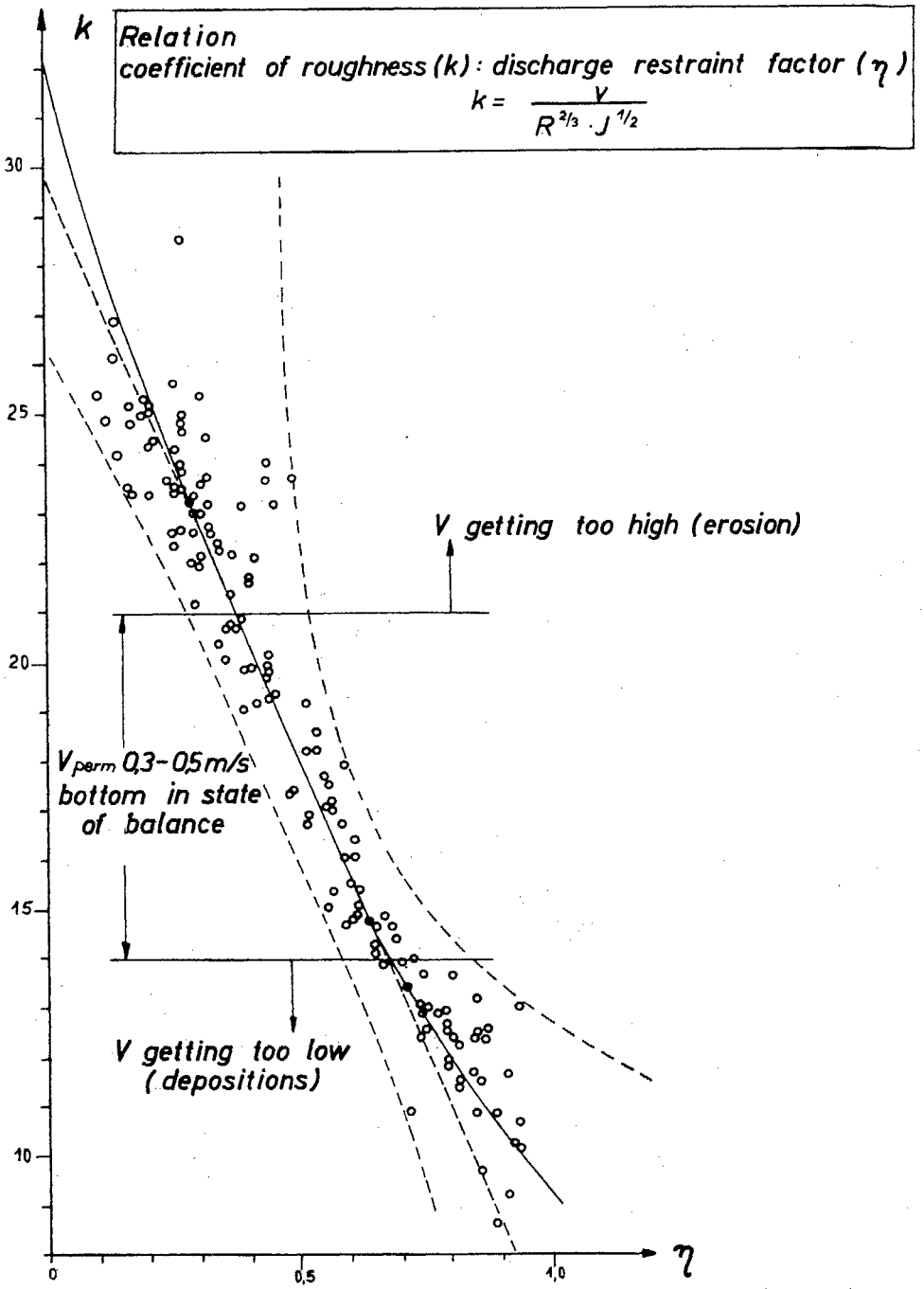


Figure 7. *Gauging station Bienenbüttel relation k: η*

La perche de jaugeage à intégration "AGAR"

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Résumé. Cette perche constitue un nouveau matériel appelé à remplacer l'équipement actuel pour la mesure des débits de basses eaux et d'eaux moyennes.

Elle permet d'obtenir la vitesse moyenne de l'eau à chaque abscisse, par "integration directe", en déplaçant un moulinet de bas en haut suivant une verticale ; la vitesse ascensionnelle est constante et contrôlée par un tachymètre.

L'appareil est électrique et semi-automatique, le mouvement étant assuré par un petit moteur de 9 W fonctionnant sur piles ordinaires.

Un compteur gradué en centimètres donne par lecture directe la profondeur de l'eau à chaque abscisse.

Un seul interrupteur permet à l'opérateur de déclencher simultanément le chrono-compteur d'impulsions et le mécanisme de l'appareil.

Le temps nécessaire pour effectuer un jaugeage est d'environ la moitié et souvent le tiers de celui nécessité par la méthode classique.

Pour le dépouillement, les graphiques et les planimétrages sont supprimés. Tout le calcul peut être fait arithmétiquement en conservant la précision des jaugeages classiques. Les résultats peuvent être rapidement obtenus sur le terrain.

THE "AGAR" INTEGRATING FLOW GAUGE

Abstract. This new apparatus is destined to replace the existing device for measuring the flows of deep or semi-deep waters. With it the average speed of the water at each abcissa can be ascertained by "direct integration", by moving an impulse spiral blade vertically from bottom to top at a constant speed, which is checked by a speedometer. This device is electrical and semi-automatic, being powered by a small 9 W motor connected to the usual batteries. A meter, graduated in centimetres, shows by direct reading the water-depth at each abcissa. One switch allows the operator to set the chronometer, the revolution counter and the mechanism of the apparatus in motion, and the time required for a reading is about one-half and often one-third of that taken by usual methods.

No diagrams and planimetering are necessary to calculate the results. This can be done arithmetically, and the results can be obtained quickly on the spot.

MEDIDAS POR EL INTEGRADOR "AGAR"

Resumen. Este mecanismo es un nuevo aparato destinado a sustituir el dispositivo existente para medir los caudales en aguas bajas o medias.

Permite obtener la velocidad media del agua en cada abscisa por "integración directa", moviendo una rueda de paletas de impulso desde el fondo a la parte superior verticalmente; la velocidad ascensional es constante y se comprueba con un medidor de velocidad.

El dispositivo es eléctrico y semiautomático, asegurándose el movimiento por un pequeño motor de 9W conectado a baterías normales y un medidor graduado en centímetros muestra la lectura directa de profundidad de agua para cada abscisa.

Un interruptor permite al operador poner en movimiento simultáneamente el cronómetro, el contador de revoluciones y el mecanismo del aparato. El tiempo que se tarda en realizar un aforo viene a ser de la mitad y con frecuencia de un tercio del que se tarda según los métodos usuales.

Para obtener los resultados, no son necesarios ni diagramas ni planimetría. Todo el cálculo se puede realizar aritméticamente, pero manteniendo la precisión de la calibración normal, además los resultados se pueden obtener con rapidez en el mismo lugar en que se realiza la calibración.

НОВЫЙ ПРИБОР ДЛЯ ИЗМЕРЕНИЯ РАСХОДОВ ВОДЫ « AGAR-2 »

Резюме. Этот шест есть новое приспособление, предназначенное для замены существующего устройства для измерения расходов низких или средних вод.

Оно позволяет получит среднюю скорост воды на каждой абсциссе путем « непосредственного интегрирования », перемещением импульсной вертушки вертикально от дна к поверхности; скорость подъема постоянна и проверяется спидометром.

Устройство электрическое и полуавтоматическое; движение создается небольшим мотором мощностью 9 ватт, соединенным с обычными батареями.

Штанга, градуированная сантиметровыми делениями, непосредственно коказывает глубину воды в каждой абсциссе.

Один выключатель позволяет оператору привести в движение одновременно хронометр, счетчик оборотов и механизм прибора.

Время, необходимое для производства измерения, составляет 1/2, а часто 1/3 времени, требуемого для обычных методов.

Результаты в виде графиков и планиметрирования исключены. Все вычисление делается арифметически, но сохраняется точность обычного измерения. Результаты могут быть быстро получены там, где производилось измерение.

L'appareil que nous allons décrire permet d'effectuer des mesures au moulinet. Nous savons qu'il existe deux méthodes que nous ne ferons que rappeler pour mémoire, l'objet de l'exposé étant surtout relatif à l'appareillage employé.

METHODE PAR POINTS

Cette méthode, qu'on utilise depuis la mise en service des moulinets, n'a pratiquement pas évolué. Le moulinet est fixé au support et la vitesse de l'eau est mesurée en au moins cinq points qu'on répartit en fonction de la profondeur entre la surface et le fond. Les débits sont calculés par double intégration, qu'on fait en pratique par voie graphique, ce qui nécessite des planimétrages.

METHODE PAR INTEGRATION DIRECTE

Elle consiste à déplacer les moulinets à vitesse constante suivant chaque verticale afin d'obtenir directement la vitesse moyenne.

Nous n'entrerons pas ici dans la démonstration mathématique de cette méthode, qu'on trouve, comme la précédente, dans les manuels d'hydrologie.

Cette disposition est très avantageuse; elle permet de réduire considérablement la durée des relevés, mais elle réduit de manière plus sensible encore la durée des travaux de dépouillement.

En pratique, on enregistre le nombre total de tours d'hélice, soit N, pendant la durée T de parcours du moulinet à une *vitesse constante* le long d'une verticale de mesure correspondant à une profondeur H.

Le déplacement vertical du moulinet pendant la mesure est obtenu soit en montant, soit en descendant ; dans le premier cas, le mouvement est assuré par un

moteur à vitesse constante ; dans le second, un régulateur centrifuge règle la vitesse de descente du moulinet lesté par un saumon.

Les divers matériels en service à ce jour ne peuvent être utilisés qu'à partir d'une passerelle, d'un pont, d'un bateau, ou d'un téléphérique, en raison de leur encombrement et de leur poids.

Il est donc impossible avec le matériel classique d'effectuer par intégration des jaugeages en cuissardes et notamment au micromoulinet. Et pourtant ce sont ces jaugeages qu'on a à faire le plus souvent.

LA PERCHE DE JAUGEAGE A INTEGRATION "AGAR-2"

La perche "AGAR" vient combler un vide jusqu'ici regrettable dans le matériel de jaugeage. Elle est spécialement étudiée pour les mesures effectuées en cuissardes, ainsi qu'à partir d'une passerelle ou d'un bateau, et permet d'obtenir la vitesse moyenne de l'eau à chaque abscisse, par *intégration directe,* en déplaçant le moulinet de bas en haut suivant une verticale (fig. 1).

La vitesse ascensionnelle est *constante,* grâce à la régularité de la source d'énergie fournie par des piles et contrôlable à chaque instant par un tachymètre.

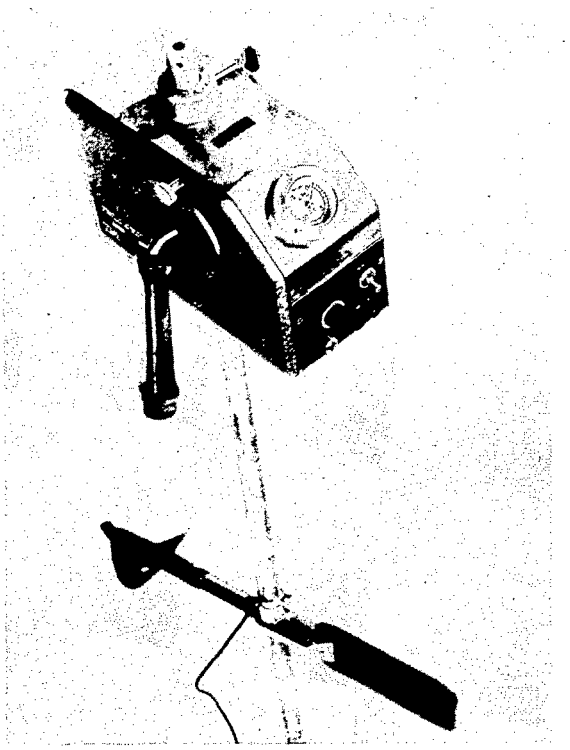


Figure 1. *Boîtier monté sur perche avec moulinet*

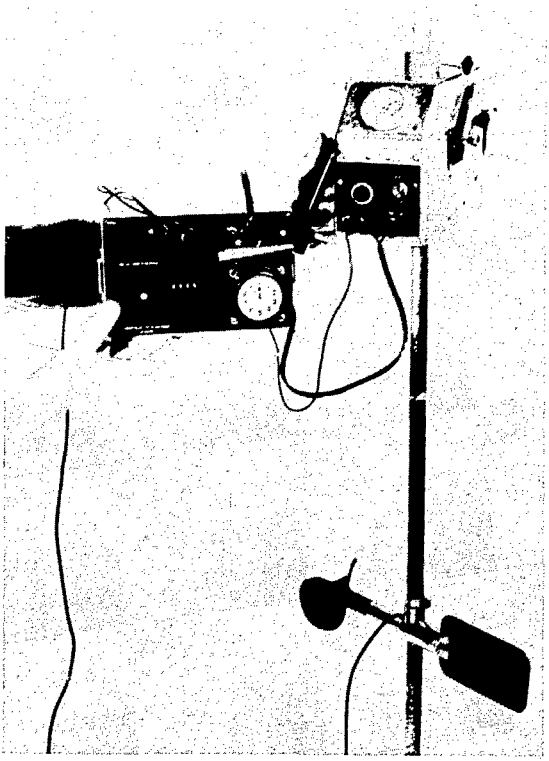


Figure 2a. *Perche montée, équipée du moulinet et du chrono-compteur NEYRPIC*

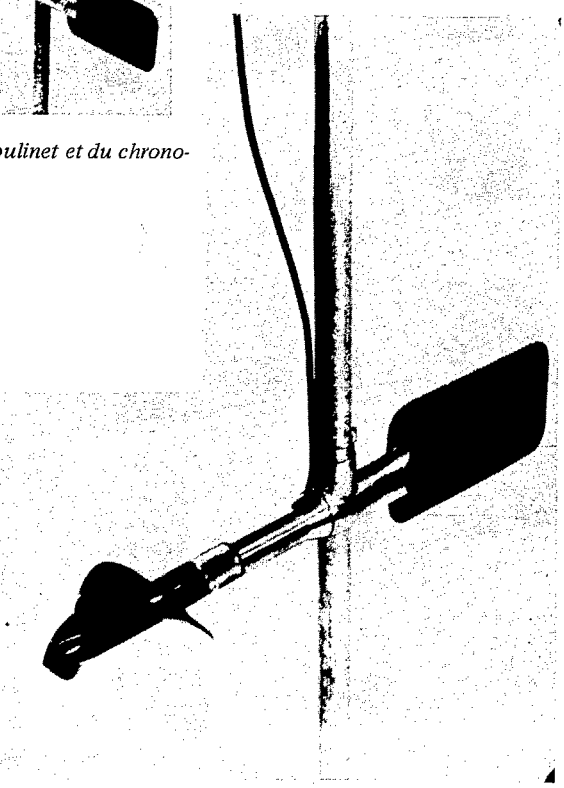


Figure 2b. *Moulinet monté sur la perche*

L'appareil est *électrique* et *semi-automatique :* le mouvement est assuré par un petit moteur de 9 W fonctionnant sur piles ordinaires (4 éléments type torche de 1,5 V) ; leur durée de fonctionnement est sensiblement la même que pour une lampe de poche.

Un réducteur ramène la vitesse à la valeur maximale d'utilisation de 15 mm/s.

Les vitesses de l'eau (V) peuvent être mesurées à partir de 0,03 m/s.

Pour 0,03 m/s $< V <$ 0,15 m/s, un rhéostat permet de régler le mouvement.

Pour $V >$ 0,15 m/s, la montée s'effectue à la vitesse maximale (15 mm/s).

La perche se compose : (a) d'une tige en duralumin de section circulaire (diamètre : 20 mm) de 3 m de longueur totale, démontable en éléments de 1 m ; elle peut être utilisée avec le nombre d'éléments désirés ; (b) d'un boîtier contenant le mécanisme, pouvant être fixé à la hauteur convenable sur la perche (fig. 2 *a* et *b*).

Le moulinet est relié au mécanisme du boîtier par un ruban perforé en métal inoxydable et il se maintient toujours dans l'axe de l'écoulement grâce à une petite dérive.

Le ruban s'enroule dans un magasin par l'intermédiaire d'une roue à picots.

La montée est toujours effectuée au moteur ; la descente se fait sous l'effet du poids et est facilitée par un débrayage qui libère le mécanisme.

Un système combiné d'embrayage et de frein permet d'assurer l'enroulement et le déroulement corrects du ruban.

Un dispositif de sécurité évite la détérioration du mécanisme en cas de blocage dans le mouvement de montée du moulinet.

Un compteur gradué en centimètres donne par lecture directe la profondeur de l'eau à chaque abscisse.

Le moulinet est d'un type courant et doit comporter une pièce avec un trou cylindrique (diamètre : 20,5 mm) permettant de l'adapter à la perche. Il doit donner un contact par tour, les temps et les impulsions étant mesurés par les appareils classiques.

Tous les compteurs peuvent facilement, moyennant une petite modification, être branchés au boîtier "AGAR", qui comporte une prise à cet effet. Il existe des chrono-compteurs qui permettent,à l'aide d'un seul interrupteur, d'actionner simultanément la montée du moulinet, le chronomètre et le compteur d'impulsions, assurant ainsi une synchronisation parfaite.

INTERET DU MATERIEL ET DE LA METHODE PROPOSES

Légère et parfaitement maniable (la perche de 2 m équipée du boîtier pèse 5 kg environ), cette perche permet d'utiliser, moyennant l'adaptatation d'une dérive, la plupart des moulinets et micro-moulinets en service (fig. 3).

Sur le terrain. L'appareillage est monté instantanément ; le temps nécessaire pour effectuer un jaugeage est d'environ la moitié et souvent le tiers de celui mis par la méthode classique ; la rapidité d'exécution permet d'explorer entièrement la section en augmentant le nombre des abcisses.

Au bureau. Les graphiques et les planimétrages sont supprimés ; tous les calculs peuvent être faits arithmétiquement en conservant la précision des jaugeages classiques ; de nombreux tests ont été effectués : sans machine à calculer, le temps nécessaire pour le calcul d'un jaugeage est de l'ordre de 10 minutes ; actuellement, de

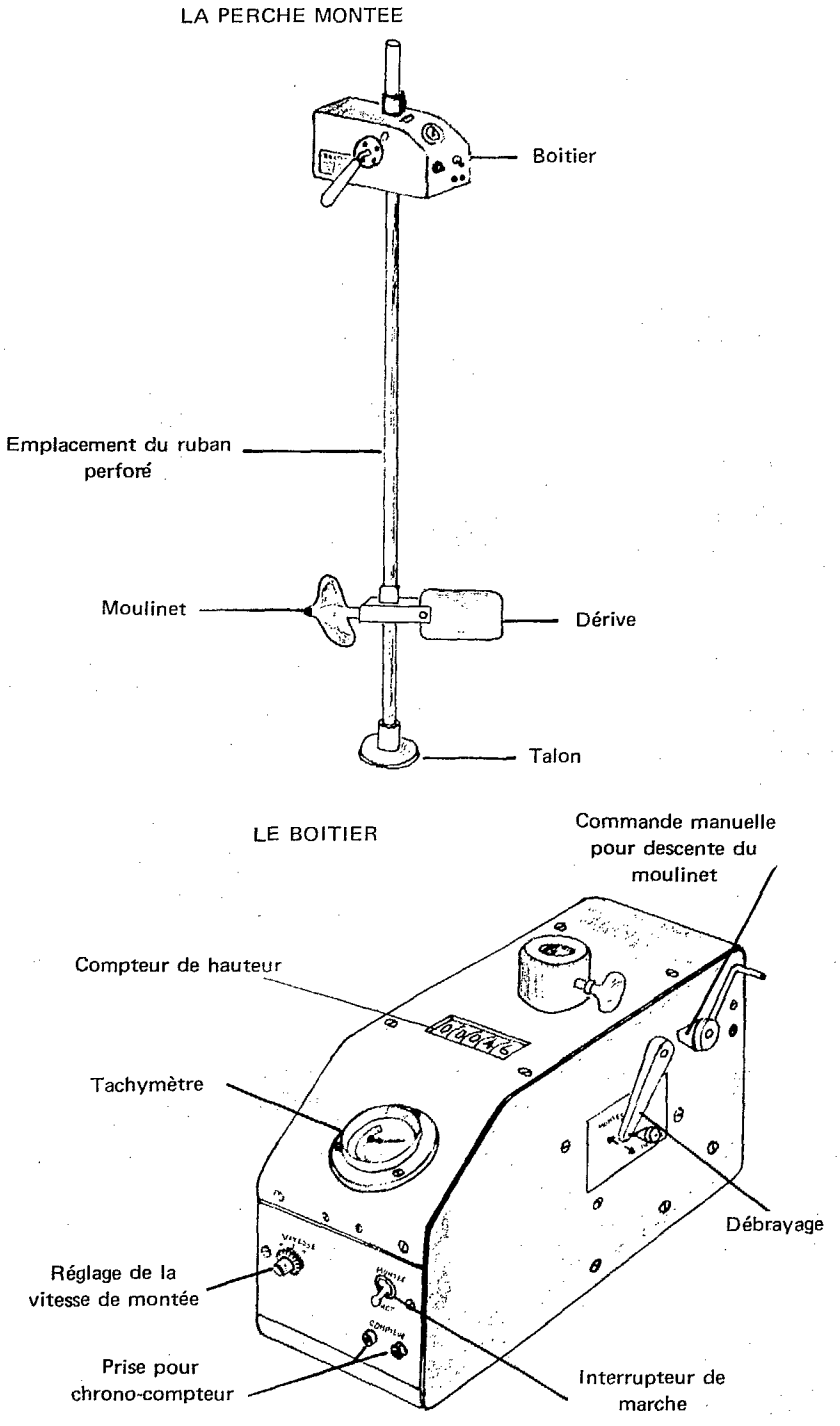


Figure 3. *Perche de jaugeage à intégration "AGAR-2", électrique, semi-automatique*

nombreux services possèdent des petites machines électroniques qui permettent, en utilisant un programme extrêmement simple, d'effectuer le calcul en 2 minutes environ.

N'importe quel opérateur, sans aucune formation, peut facilement utiliser la perche à intégration et calculer les jaugeages, même sur le terrain.

Par la rapidité d'exécution qu'elle permet, elle est parfaitement adaptée pour les mesures à effectuer dans des cours d'eau ou canaux, dont la cote du plan d'eau varie très vite (étude des résurgences, mesure des effluents industriels, etc.).

La perche "AGAR-2" remplace désormais le matériel classique pour toutes les mesures de débits pouvant être effectuées en cuissardes.

DISCUSSION

Dosseur. Je signale quelques améliorations apportées à l'appareil de M. Duboë et qui facilitent la réalisation pratique des jaugeages à l'aide de cette perche : (a) adaptation d'un compteur d'impulsion directement sur le boîtier et d'un commutateur assurant le synchronisme parfait des trois opérations de mesure : mouvement de montée du moulinet, comptage des impulsions et mesure du temps ; (b) suppression des liaisons par câbles entre moulinet et compteur : la perche est rendue conductrice.

L'ensemble est ainsi entièrement autonome et le jaugeage est très aisé.

Flow measurement by the integrating float method

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Abstract. It has long been recognized that a float released at the bottom of a stream may be used to determine the discharge of the stream. This method, called "the integrating float technique", gives discharges for any kind of velocity profile. It is particularly suited for streamflow measurement at low velocities, but not limited to it. Other advantages include accuracy (when used properly), simplicity, and the lack of need for calibrations. In this paper, the writer reports some recent studies and discusses certain measuring problems.

MESURE DE DEBIT PAR LA METHODE DU FLOTTEUR INTEGRATEUR

Résumé. Il est connu depuis longtemps qu'un flotteur abandonné au fond d'un courant peut être utilisé pour déterminer le débit du courant. Cette méthode, appelée "technique du flotteur intégrateur", donne les débits pour toute espèce de profil de vitesse. Elle est particulièrement adaptée à la mesure du débit aux faibles vitesses, mais n'est pas limitée à ce cas. Ses autres avantages sont la précision (quand elle est correctement utilisée), la simplicité, et le fait qu'elle ne nécessite pas d'étalonnage. Dans le présent article, on rend compte des derniers développements de cette technique et l'on discute de certains problèmes de mesure.

MEDICIÓN DEL CAUDAL POR EL MÉTODO DEL FLOTADOR INTEGRADOR

Resumen. Desde hace tiempo se ha admitido que un flotador abandonado en el fondo de una corriente se puede utilizar para determinar el caudal de la misma. Este método, llamado "técnica del flotador integrador", da los caudales para todo tipo de perfil de velocidad. Se adapta particularmente bien a la medida del caudal con velocidades pequeñas, pero no se limita a estos casos. Sus otras ventajas son la precisión (cuando se utiliza correctamente), la simplicidad y el hecho de que no necesita calibración. En la presente comunicación se da cuenta de los últimos progresos en esta técnica y se discuten determinados problemas de su utilización.

ИЗМЕРЕНИЕ РАСХОДА МЕТОДОМ ИНТЕГРИРУЮЩЕГО ПОПЛАВКА

Резюме. Уже давно известно, что поплавок, освобожденный у дна потока, может быть использован для измерения расхода потока. Этот способ, названный « способом интегрирующего поплавка », дает расходы для любого вида скоростного профиля. Он особенно применим для измерения расходов при малых скоростях, но не ограничивается только этим. Другие преимущества его включают точность (при правильном применении), простоту, отсутствие необходимости в тарировке. В докладе автор сообщает о ряде недавних исследований и рассматривает некоторые вопросы методики измерений.

INTRODUCTION

For centuries, men have tried to utilize floats of various kinds for the measurement of velocity and discharge in streams. These include surface, subsurface, and rod floats. Although float measurements are generally easy to make and do not need sophisticated and expensive equipment, they have the common drawback of being inaccurate. Error arises mainly from the difficulty in relating the float velocity to the mean (depth-averaged) velocity of the flow. As the shape of the vertical profile of velocity varies from one situation to another, a general relationship between the float velocity and the mean velocity is intractable. The integrating float method does not have such a problem.

In 1882, d'Auria [ref. 1] concluded from an analysis that when a heavy sphere is released from the surface of a stream, the horizontal distance it travels during its fall in water, divided by the time of fall, is equal to the mean velocity of the flow. From this he proposed the use of a tethered, spiked, and heavy sphere to measure mean velocity (tethered so that the sphere can be retrieved and the distance travelled can be measured, and spiked and heavy so that it would not roll upon arrival at the stream bottom). It is interesting to note that although d'Auria seems to be the first to have proposed the use of a descending object to measure mean velocity, he erred in his derivation in that he averaged the velocity profile with respect to time instead of vertical distance. As pointed out by the writer in an earlier publication [2] a descending or ascending object gives correct mean velocity only when it falls or rises at a constant speed. Since a heavy sphere such as that proposed by d'Auria accelerates during its fall, it cannot yield accurate mean velocity except for flows with uniform velocity.

Hajos [3], in 1904, measured discharges using floats released from bottoms of streams. He also proposed a double-float technique to be used when the location from which a float is released cannot be determined with accuracy. In 1910, Bureau [4] reported on a similar technique to determine mean velocity; but his idea is different from the integrating float technique in that it requires the measurement of the entire trajectory of the float during its rise. Miyagi [5], 1929, used compressed air to determine discharges. His idea is the same as the integrating float method except for the use of air bubbles as floats. Other known contributors to the integrating-float technique include Wells [6] 1943, Taylor [7], 1951, Viol and Semenov [8], 1964, Rusakov [9], 1964, Martin [2, 10, 11]. 1966, Morris [12, 13], 1968, and Liu [2, 11, 13].

DISCUSSION OF BASIC EQUATIONS

If a float released from the bottom of a stream or canal travels a horizontal distance L during its rise, it can be proved that [2, 11] the discharge per unit width, q, and the mean velocity, V, are

$$q = \frac{L}{t_0} D \tag{1}$$

$$V = \frac{L}{t_0} \tag{2}$$

In which t_0 is the rise time, i.e. the time it takes the float to reach water surface, from the time it was released; and D is the flow depth. The derivation of equations 1 and 2 is based on: (a) steady flow, (b) uniform flow, (c) infinitesimally small float, (d) the float reaches the horizontal velocity of the flow and a constant upward velocity instantaneously after release, (e) absence of secondary currents, and (f) absence of turbulence. Detailed consideration of these conditions has been reported before [2, 11, 13]; only the important results will be recapitulated herein.

The steady flow requirement can be met when the discharge in a stream varies little over the time needed to complete measurements. The uniform flow requirement can be met by choosing a relatively straight and uniform reach of channel, and by using a relatively fast-rising float. As far as the float size is concerned, a float with a diameter smaller than $0.02D$ will cause less than 1 per cent error.

The requirement that a float must have the same horizontal velocity as that of the fluid and a constant upward velocity can be fulfilled approximately by using a float that approaches terminal velocity rapidly. When a float is chosen such that $t_0/t_6 \geqslant 100$, where t_6 is the time to reach 60 per cent terminal velocity, error introduced from this source is less than 1 per cent. The value of t_6 for any float can be evaluated from Fig. 1, in which $T_6 = t_6\, v/d^2$, where v is the kinematic viscosity of the fluid and d is the float diameter; S is the density ratio, i.e., the density of float to that of the fluid; K is 0.5; and $R_T = V_T\, d/v$, where V_T is the terminal velocity of the float which can be found through the use of a graph for drag coefficient of spheres given in most fluid mechanics texts. Using such a procedure, it can be shown that [2] for a 1.9 g ping-pong ball 3.8 cm in diameter, $t_0/t_6 = 100$ corresponds to a water depth of approximately 6 metres. Thus accurate measurements of discharges by ping-pong balls are possible only when the flow depth exceeds approximately 6 metres.

A quantitative analysis of the effect of secondary currents on the integrating float method is not yet available. But the influence is thought to be small in wide and straight channels. Furthermore, the use of a fast-rising float also helps to minimize the secondary-current effect. The influence of turbulence will be discussed in the next section.

When a float is chosen such that $t_0 \geqq 100\, t_6$, as recommended before, the quantity D/t_0 in equation 1 becomes essentially the same as the terminal velocity V_T. This suggests that q can be found from a sole measurement of L, once the value of V_T is known. However, since the value of V_T for a given float is not always constant (it depends on factors such as the density of water, turbulence, etc.), it is inaccurate to use the same V_T in different flows. As a matter of fact, even in the same flow the value of V_T may not be the same for different releases. This points to the need for measurements of the quantities L, t_0, and D in order to obtain accurate results.

When a float is released from a device suspended on a cable in a flow of high velocity, the exact location of the release device on the channel bottom may not be known. Thus the quantity L cannot be determined accurately. In such a situation, the double float technique proposed by Hajos may be used. By releasing two floats of quite different terminal velocity from the same location, and by measuring the differences of L's and t_0's, q can be found from

$$q = \frac{L_2 - L_1}{(t_0)_2 - (t_0)_1} \,. \tag{3}$$

Equation 3 is based on the same assumptions that equations 1 and 2 are based on.

EFFECT OF TURBULENCE

Consider a float released N successive times from a given location on a channel floor. When the flow is turbulent, quantities L and t_0 for each release will be different. Thus the discharge per unit width, q, as calculated from equation 1 will be different from one release to another. Taking the arithmetic mean of equation 1 with respect to the N releases,

$$\bar{q} = D\left(\frac{\bar{L}}{t_0}\right) \tag{4}$$

where the bar above each quantity denotes the arithmetic mean of the quantity. Equation 4 can be used to calculate the discharge per unit width of the flow. It is not difficult to see that the larger the number N used in the calculation, the closer q is to the true discharge per unit width.

When the variations of individual values of L and t_0 from their corresponding means are small,

$$\left(\frac{\bar{L}}{t_0}\right) \doteq \frac{\bar{L}}{\bar{t}_0}. \tag{5}$$

For instance, Martin [10] released each float 10 times at a fixed location. An evaluations of his data showed that equation 5 errs much less than 1 per cent for all the 40 runs taken [11]. Combination of equations 5 and 4 yields

$$\bar{q} = D\frac{\bar{L}}{\bar{t}_0}. \tag{6}$$

Likewise, one can prove that

$$\overline{q^2} = D^2 \cdot \frac{\overline{L^2}}{\overline{t_0^2}}. \tag{7}$$

Substituting $L = \bar{L} - L'$, $t_0 = \bar{t}_0 - t_0'$, and $q = \bar{q} - q'$ into equation 7, where the primed quantities denote variation from their arithmetic means, and using equation 6 and the relation $\bar{L}' = \bar{t}_0 = \bar{q}' = 0$,

$$q_{CV} = \sqrt{\frac{L_{CV}^2 - (t_0)_{CV}^2}{1 + (t_0)_{CV}^2}} \tag{8}$$

where the subscript "CV" denotes the coefficient of variation, i.e. the standard deviation divided by the mean of the same quantity.

Equation 8 relates the three statistical quantities q_{CV}, L_{CV}, and $(t_0)_{CV}$. It indicates that L_{CV} is greater than $(t_0)_{CV}$. The equation is strictly correct only when the number of releases, N, is infinitely large. Otherwise, it should be considered only as an approximation.

The coefficient of variation is an index for the average relative deviation of a quantity from its mean. It follows then that the larger the coefficient of variation of a certain quantity, the larger the expected error of the quantity obtained from a single measurement, and also the larger the error of the mean value obtained from a given number of measurements. For instance, from statistics it is known that by

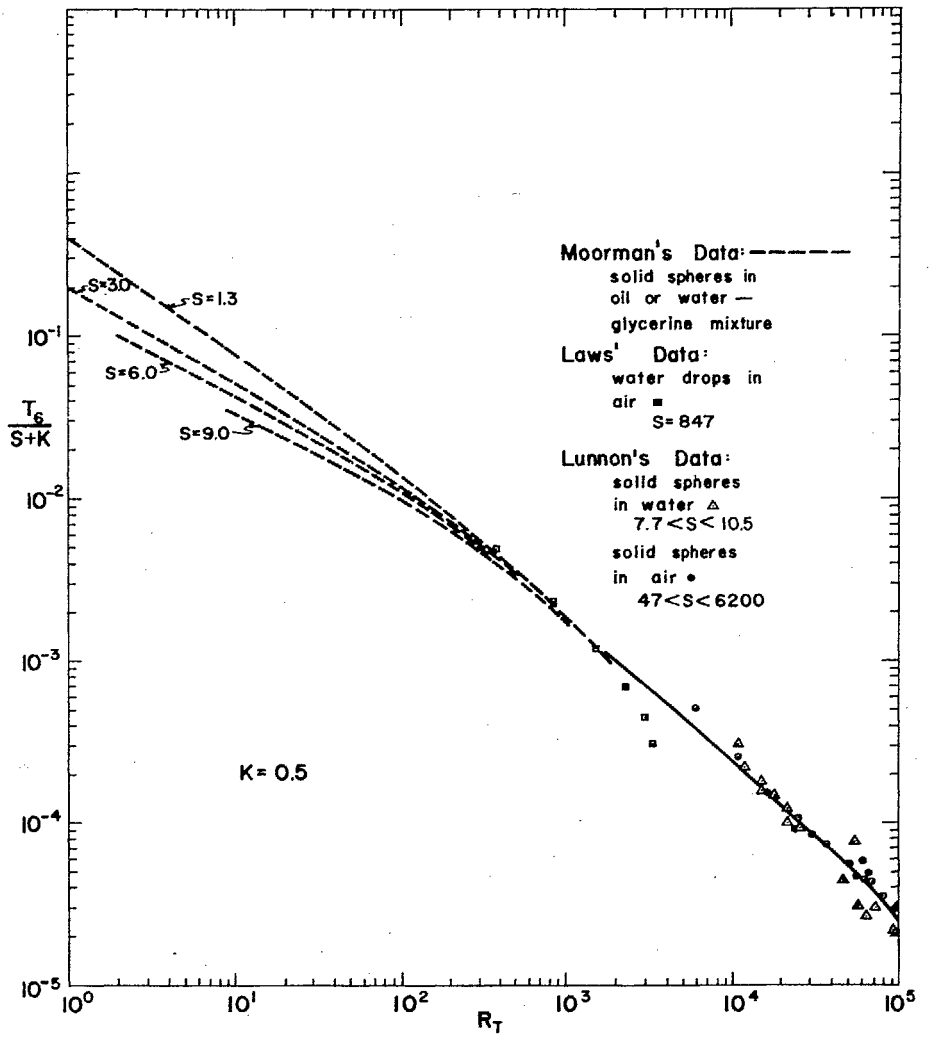


Figure 1. *Plot of* $T_6/S+K$ *versus* R_T

assuming a Gaussian distribution for q, 32 per cent of q are expected to have an error greater than q_{CV}, 5 per cent are expected to have an error greater than $2q_{CV}$, and only 0.3 per cent err more than $3q_{CV}$. Also from statistics, the coefficient of variation of $\overline{q}$, based on N releases, is

$$(\overline{q})_{CV} = \frac{q_{CV}}{\sqrt{N}}. \tag{9}$$

Equation 9 can be used to calculate the expected error of $\overline{q}$, from a knowledge of q_{CV}.

To relate q_{CV} to turbulence characteristics, one can proceed as follows: consider the float released repeatedly at a given location on channel bottom; if the vertical distance travelled by the float in a time t is y, application of Taylor's theory on turbulent diffusion yields [14]:

$$y_{sd} = \sqrt{\overline{u_y'^2}} \cdot \sqrt{2I_y} \cdot \sqrt{t}. \tag{10}$$

In equation 10, y_{sd} is the standard deviation of y for a large number of releases; u_y' is the y-component of turbulence; and I_y is a Lagrangian time scale with respect to y. Equation 10 is based on three assumptions: (a) homogeneous turbulence, (b) infinitesimally small float, and (c) long diffusion time.

The standard deviation of y can be related to the standard deviation of t for a constant y by

$$t_{sd} = t\frac{y_{sd}}{y}. \tag{11}$$

Substitution of equation 10 into equation 11 yields

$$(t_0)_{sd} = \sqrt{\overline{u_y'^2}} \cdot \sqrt{\frac{2I}{2I_y}} \cdot \frac{\overline{t}_0^{3/2}}{D} \tag{12}$$

and

$$(t_0)_{CV} = \sqrt{\overline{u_y'^2}} \cdot \sqrt{2I_y} \cdot \frac{\sqrt{\overline{t}_0}}{D} \tag{13}$$

The magnitude of L for each release is influenced by all the three components of turbulence. The x-component of turbulence, u_x', causes a turbulent diffusion in the longitudinal direction that can be calculated from Taylor's theory. The result is

$$(L_1)_{sd} = \sqrt{\overline{u_x'^2}} \cdot \sqrt{2I_x} \cdot \sqrt{\overline{t}_0}. \tag{14}$$

The y-component of turbulence, as discussed before, causes a variation in t_0. This in turn makes L to vary according to

$$(L_2)_{sd} \doteq V(t_0)_{sd} \tag{15}$$

The z-component of turbulence causes a lateral movement of a float which can also change the value of L when there is a lateral variation of mean velocity (dispersion). However, since this dispersion is small when a fast-rising float is employed in a wide, open channel, it is to be neglected herein.

Assuming little correlation between L_1 and L_2, combination of equations 14 and 15 yields

$$L_{CV} = \sqrt{2\bar{t}_0 I_x \overline{u_x'^2} \frac{1}{L^2} + (t_0)_{CV}^2}. \tag{16}$$

Equation 16 reveals that L_{CV} is greater than $(t_0)_{CV}$, confirming a result obtained from equation 8.

Combination of equations 8, 13, and 16 yields

$$q_{CV} = \sqrt{\frac{2\bar{t}_0 I_x \overline{u_x'^2}}{L^2 [1 + (t_0)_{CV}^2]}}. \tag{17}$$

Under ordinary circumstances, $(t_0)_{CV}^2 \ll 1$, thus equation 17 reduces to

$$q_{CV} = \sqrt{\frac{\overline{u'^2}}{V}} \cdot \sqrt{\frac{2I_x}{\bar{t}_0}}. \tag{18}$$

Note that by virtue of $(t_0)_{CV}^2 \ll 1$, the effect of u_y' on q_{CV} has been eliminated.

This means that what mainly affects q is u_x', a result that was derived in ref. 13 from a more intuitive approach. It may be concluded from equation 18 that q_{CV} is directly proportional to the relative turbulence intensity, and the square-root of a time scale of turbulence, and inversely proportional to the square-root of $\bar{t}_0$.

Since the above derivation is based on a homogeneous turbulence field, and since in an ordinary open channel the turbulence is inhomogeneous in the y-direction, equation 18 should be considered as an approximation only. Besides, the values of $u_x'^2$ and I_x in an open channel represent values averaged over the entire depth of the flow.

REMARKS AND SUMMARY

The integrating float method is basically different from ordinary float and current meter methods in that the float used herein continuously "integrates" the vertical velocity-profile as it rises. Consequently, proper application of the method yields accurate discharge of any type of velocity profile. Another special characteristic of the technique is that its accuracy does not depend on the magnitude of velocity, hence making accurate measurements possible at velocities much below those that can be measured with conventional current meters [11]. A third feature is that there is no need for meter calibration such as is required for current meters.

It should be clear from the earlier discussion that for accurate determination of q in a turbulent flow, a float must be released many times at a fixed location. However, as far as the total discharge in a wide, open channel is concerned, accurate determination can be achieved with only one or two releases at each location, provided that the float is released from a number of points (say, 20) across the channel bottom [13].

Notwithstanding some promising features, the technique cannot yield accurate results unless used properly, and unless quantities L, t_0, and D can be determined with precision. For this reason, recommendations outlined in this paper as well as in ref. 2 should be followed in using the integrating float method. Moreover, a measuring technique must be developed for convenient and precise measurements of L and t_0 in the field. In the writer's opinion, a system consisting of a movie camera, an

electric timer, and an electrically controlled float-release device such as that used in a previous laboratory study [12] can be modified to fit field conditions. Tasks remain in the development and perfection of such a system, and in the experimentation and use of it in streams.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. D'Auria, L. Velocity of streams. A new method to determine correctly the mean velocity of any perpendicular in rivers or canals. *(The) American Engineers*, vol. 3, no.7, Feb. 1882.
2. Liu, H., Martin, L. D. Analysis of integrating-float flow measurement. *Journal of the Hydraulics Division,* ASCE, vol. 94, no.HY5, Sept. 1968.
3. Hajos, S. Neues Verfahren zur Messung kleiner Wassergeschwindigkeiten, *Zentralblatt der Bauverwaltung,* vol. 24, no.44, 1904.
4. Bureau, H. Nouvelle méthode de jaugeage par flotteurs. *La Houille Blanche,* vol. 9, no 5, May 1910; vol. 9, no.6, June 1910; vol. 9, no.7, July 1910; vol. 9, no.9, Septembre 1970.
5. Miyagi, 0. Measurement of Stream Velocity by Air Bubbles. *Technology Report of the Tohoku Imperial University,* vol. 8, no. 4, Sandai, Japan, 1929. Also: *Journal of the Society of Mechanical Engineers (Japan),* vol. 32, no. 147, July 1929.
6. Wells, J. V. B. Measurement of low flow in canalized streams. *Water Resources Bulletin, USGS,* Washington, Aug. 1943.
7. Taylor, E. J. Low-velocity measurement. *Water Resources Bulletin, USGS,* Washington, Nov. 1951.
8. Viol, V. E.; Semenov, V. I. Experiment in measuring discharges in canals by the photo-integration method. *Soviet Hydrology, Selected Papers,* no. 2, 1964.
9. Rusakov, V. E. Investigations of liquid integrating floats in streamflow. *Soviet Hydrology, Selected Papers,* no. 4, 1964.
10. Martin, L. D. Flow measurement at low velocity by the integrating-float method. M. S. Thesis, University of Missouri-Columbia, April 1967.
11. Liu, H.; Martin, L. D. Integrating-float measurements at low velocities. *Journal of the Hydraulics Division, ASCE,* vol. 96, no. HY1, Jan. 1970.
12. Morris, C. D. Flow measurement by integrating-float method in turbulent flows. M.S. thesis, University of Missouri-Columbia, June 1968.
13. Liu, H.; Morris, C. D. Integrating-float measurements in turbulent flows. *Journal of the Hydraulics Division, ASCE,* vol. 96, no.HY2, Feb. 1970.
14. Taylor, G. I. Diffusion by continuous movements. *Proceedings, London Mathematical Society,* series 2, vol. 20, 1921.

DISCUSSION

Rolff: What is the accuracy of this method?

Liu: All experiments are based on our laboratory studies. In the laboratory the length error and the time t can be measured precisely, both at a very low velocity and in highly turbulent flow. We found an average error of less than 3 per cent for highly turbulent flow. At lower turbulence level it is more accurate than 3 per cent. We compared this discharge measured by the integrating float method with the discharge measured by a sharp crested weir which is believed to be rather accurate.

Rodier: J'aurais été très intéressé d'obtenir des détails sur les dispositifs pratiques pour utiliser cette méthode sur le terrain. Je comprends qu'on puisse l'utiliser pour des vitesses de 2 à 5 cm/s. Je vois mal comment opérer dans le cas où nous utilisons habituellement les flotteurs, c'est-à-dire pour des vitesses supérieures à 5 cm/s.

Liu: It is true that current meters are much more conveniently used at higher velocities than this float method. But as I mentioned earlier, it is intended not to replace the current meter but to supplement it at very low velocities and in conditions where the velocity profile is known. If the direction of the flow changes across the depth, you have to take many measurements or if you use an integrated process, as the previous author indicated, you cannot simply use a 0.2. 0.8 or 0.6 method. This method is intended for use at very low velocity and in situations where current meters are difficult to use.

Kornitz: (a) The accuracy of 3 per cent is rather astonishing. It might be considered a good result for a current meter and not for a float.

(b) What was the velocity measured?

(c) What is the upper limit of velocity?

(d) What is the lower limit of depth?

Liu: The 3 per cent error is based on our study in a flume. The floater used is of a size less than 5 mm. We used a photographic technique combined with an electric timer to determine precisely the length error. The average of this depends on how accurately you can determine l and t. In the laboratory it is possible to determine it very precisely. We released it at nine locations across the flume, twice at each location. All together it is based on eighteen measurements across the flume. We even put a screen across the flume to change the level of turbulence. The turbulence intensity is around 20 to 30 per cent. The 3 per cent error was compared to the discharge measured by a sharp crested weir.

Struijk: What is the size of the float?

Liu: In the laboratory it varies. The maximum size was about 1 cm diameter.

Graze: In line with the previous speaker's query, I should also like to question the 3 per cent accuracy stated for the integrating float method. Since the method depends on a rapid attainment of the terminal velocity of the float, the error of only 1 per cent or less as reported for this aspect, provided the criterion of $t/t_6 \geqq 100$ is adhered to, is especially queried.

I should like to ask the author whether he would care to: (1) comment again on this stated 1 per cent accuracy, especially on how it was determined and what laboratory tests, if any, were undertaken to check its validity, and (2) indicate the difficulties of maintaining this "$t/t_6 \geqq 100$" criterion when in fact "a relatively fast-rising float" is a further stated specification.

Liu: If you make the range time t larger than 100 times t_6 which is a time constant of the float then the effect of terminal velocity will be negligible. If the velocity profile is relatively uniform you can use a float even with a much larger acceleration.

Graze: But then you would be in trouble concerning your 3 per cent accuracy because that is quite a high accuracy.

Use of depth floats in drainage canals with aquatic weed

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Abstract. A theoretical analysis of the profile of water velocities in artificial irrigation canals is presented. The analysis is aimed at the whole cross-section as well as those parts which are accessible for using depth (bar) floats. They are usually the sections occurring in the centre of the trapezoidal profile and close to the water level due to the spread of weeds over the canal bed and banks during the growing period.

As a result of this analysis, coefficients have been developed for converting velocities, recorded by means of floats, to the mean profile velocity. A comparison of the results with measurements in the field is also presented.

USAGE DES FLOTTEURS IMMERGES DANS LES CANAUX DE DRAINAGE CONTENANT UNE VEGETATION AQUATIQUE

Résumé. Le présent travail fournit une analyse théorique du champ de vitesse des écoulements dans les canaux d'irrigation artificiels. L'analyse porte sur la section transversale totale, d'une part, et, d'autre part, sur la section accessible à l'utilisation de flotteurs immergés (barre). D'habitude, la partie étudiée est le centre d'un profil trapézoïdal et est située près de la surface, en raison de la végétation aquatique couvrant le fond et les rives du canal pendant la période de croissance.

En conclusion, on donne les coefficients permettant de calculer la vitesse moyenne dans la section à partir des vitesses mesurées avec les flotteurs ; une comparaison des résultats ainsi trouvés avec les mesures effectuées sur place clôt cette étude.

UTILIZACIÓN DE LFOTADORES SUMERGIDOS EN LOS CANALES DE DRENAJE CON VEGETACIÓN ACUÁTICA

Resumen. La presente comunicación presenta un análisis teórico de las velocidades del agua en los canales artificiales de riego. Se estudia por una parte la sección transversal total y por otra la sección accesible para la utilización de flotadores sumergidos (barra). Con frecuencia esta es la parte situada en el centro de un perfil trapezoidal y próxima al nivel del agua es decir, como consecuencia de la vegetación acuática que cubre el fondo y las riberas del canal durante el periodo de crecimiento.

Como consecuencia de este análisis, se ha preparado una propuesta de coeficientes para convertir las velocidades registradas por medio de flotadores en la velocidad que caracteriza al perfil medio y también se presenta la comparación de los resultados con las mediciones directas.

ИСПОЛЬЗОВАНИЕ ГЛУБИННЫХ ПОПЛАВКОВ В ИРРИГАЦИОННЫХ КАНАЛАХ С ВОДНОЙ РАСТИТЕЛЬНОСТЬЮ

Резюме. Выполнен теоретический анализ распределения скоростей в искусственных ирригационных каналах. Этот анализ относится как ко всему сечению, так и к его части, доступной для использования глубинных поплавков. Обычно этой частью является центр трапецоидального сечения вблизи водной поверхности, вследствие зарастания дна канала и его откосов в вегетационный период.

В результате этого анализа предложены коэффициенты для перехода от скоростей, измеренных поплавками, к средним скоростям в сечении. Дано также сравнение результатов, полученных этим методом с полевыми измерениями.

INTRODUCTION

Drainage and irrigation canals are usually constructed in lowland regions. Their dimensions and slopes are relatively small, hence only low water velocities occur. This leads to the growth of aquatic weed which excludes many methods of discharge measurement.

Measurement by floats is simple and effective. In addition, the submerged float is not affected by wind and gives the mean water velocity over a certain range of depths and widths of the canal cross-section.

ASSUMPTIONS OF THE SOLUTION

A trapezoidal canal cross-section is considered with the bank slopes of $1 : m$, width at the bottom B_1, water depth in the centre of the cross-section h_v and a total width at the water level $B_1 + 2 \cdot B_2$, where

$$B_2 = m \cdot h_v. \tag{1}$$

Bottom and bank roughness are assumed to be equal (i.e. after overgrowing or cutting) and are expressed by the coefficient n.

The formula for mean profile velocity v_F is used in the form

$$v_F = \frac{1}{n} \cdot R^b \cdot I^{1/2} = a \cdot R^b \tag{2}$$

where R = hydraulic radius; I = slope of the canal bottom (and water level); b = exponent in the range from 0.5 to 0.8.

Among others, the formulae of Chézy, Strickler-Manning and Forchheimer could be used.

Considering the profile of velocities, the simplified equation 2 should be used for the mean vertical velocity v_v, in the form

$$v_v = a \,.\, h_v^b \,. \tag{3}$$

The assumptions mentioned are shown in Fig. 1.

SOLUTION

Vertical velocity

The velocity profile in the vertical is given by the expression

$$v = a \,.\, (b + 1) \,.\, h^b \tag{4}$$

chosen so as to obtain the mean vertical velocity

$$v_v = \frac{1}{h_v} \cdot \int_0^{h_v} v \cdot \mathrm{d}h \tag{5}$$

having the shape mentioned in equation 3. The depth h increases from the bottom to the water surface.

The mean velocity in the vertical segment $(1 - k) \cdot h_v$ will be

$$v_v^* = \frac{1}{(1-k) \cdot h_v} \cdot \int_{kh_v}^{h_v} v \cdot \mathrm{d}h = \frac{1}{(1-k) \cdot h_v} \cdot a \left[h_v^{b+1} - (kh_v)^{b+1} \right]. \tag{6}$$

Discharge

Discharge through the entire cross-section is given by the expression

$$Q = \int_0^B v_v \cdot h \cdot \mathrm{d}B. \tag{7}$$

In the central part of the cross-section (Fig. 1) it is:

$$Q_{\square} = a \cdot h_v^b \cdot h_v \cdot \int_0^{B_1} \mathrm{d}B = a \cdot h_v^{b+1} \cdot B_1 \; ; \tag{8}$$

in both boundary parts (Fig. 1) of the cross-section:

$$Q_{\Delta} = 2 \cdot a \int_0^{B_2} h \cdot \mathrm{d}B. \tag{9}$$

Substituting in the range from 0 to B_2

$$h = \frac{1}{m} \cdot B, \tag{10}$$

we obtain the solution of equation 9

$$Q_{\Delta} = 2a \cdot \frac{1}{b+2} \cdot \left(\frac{1}{m}\right)^{b+1} \cdot B_2^{b+2} \tag{11}$$

and equation 7 becomes

$$Q = a \cdot \left(\frac{1}{m}\right)^{b+1} \cdot \left[B_1 \cdot B_2^{b+1} + \frac{2}{b+2} \cdot B_2^{b+2} \right]. \tag{12}$$

Discharge in the part shown dashed in Fig. 1 is

$$Q^* = \int_0^{B_1} v_v^* \cdot (1-k) \cdot h_v \cdot \mathrm{d}B = a \cdot (1 - k^{b+1}) \cdot \left(\frac{1}{m} \cdot B_2\right)^{b+1} \cdot B_1. \tag{13}$$

This is the discharge which could be measured by floats without great inaccuracies.

Cross-sectional area

Including the area covered by the vegetation, i.e. when the canal has been cleaned, it is

$$F = h_v \,.\, B_1 + \frac{1}{m} \,.\, B_2^2 = \frac{1}{m} \,.\, B_2 \,.\, (B_1 + B_2)\,. \tag{14}$$

The dashed area in Fig. 1 is

$$F^* = (1 - k) \,.\, h_v \,.\, B_1 = (1 - k) \,.\, \frac{1}{m} \,.\, B_1 \,.\, B_2. \tag{15}$$

Mean velocity

In the whole cross-section, it is

$$V_F = \frac{Q}{F} = a \,.\, \left(\frac{1}{m}\right)^b \,.\, \left[B_1 \,.\, B_2^b + \frac{2}{b+2} : B_2^{b+1}\right] \,.\, (B_1 + B_2)^{-1}\,. \tag{16}$$

In the dashed area (Fig. 1), it is

$$V_F^* = \frac{Q^*}{F^*} = a \,.\, \left(\frac{1}{m}\right)^b \,.\, \frac{1 - k^{b+1}}{1 - k} \,.\, B_2^b. \tag{17}$$

Finally, the ratio of mean velocities

$$\frac{V_F}{V_F^*} = \frac{1 - k}{1 - k^{b+1}} \,.\, \frac{\dfrac{B_1}{B_2} + \dfrac{2}{b+2}}{\dfrac{B_1}{B_2} + 1} \tag{18}$$

gives the conversion factor dependent upon (a)* the choice of the velocity equation (Chézy's, Manning's, etc.); (b) the relative depth of float submergence $(1 - k)$; (c) the ratio of the width B_1 (bottom width) to B_2 (water level width above the wetted bank slope).

The variation of the function 18 with the usual value of $b = 2/3$ is shown in Fig. 2.

DISCUSSION

Inaccuracies resulting from the simplification $R = h$ at the transition from the equation 2 to 3.

1. Hydraulic radius R

$$R = \frac{F}{P} \tag{19}$$

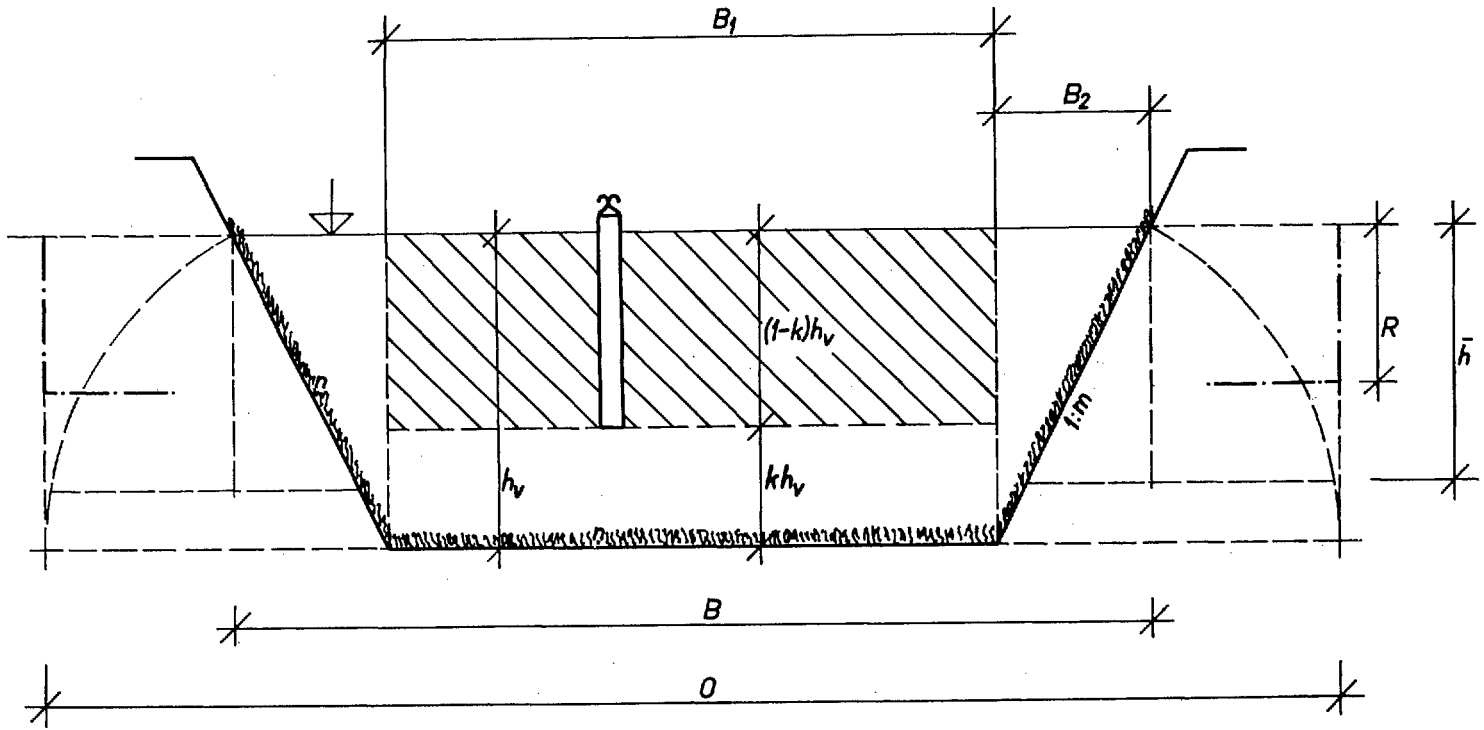


Figure 1. *Assumptions of the solution*

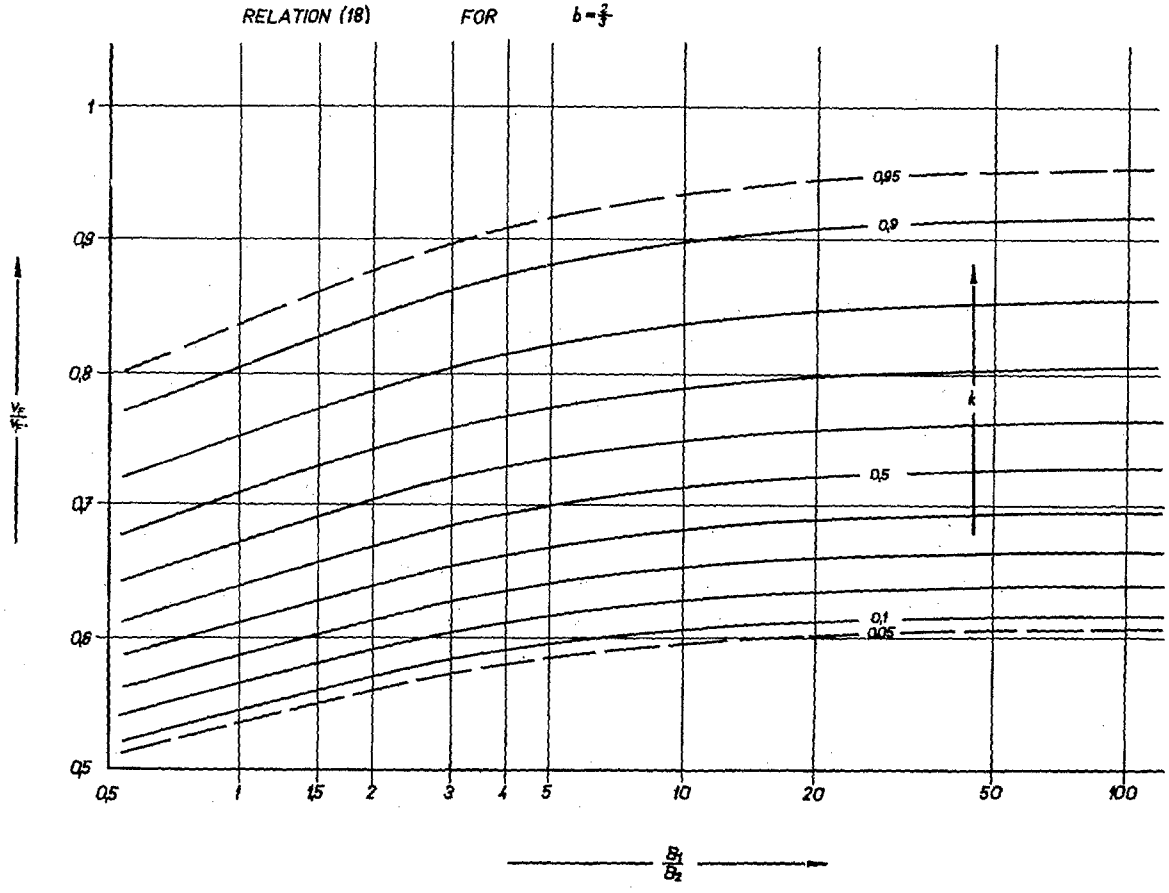


Figure 2. *Equation (18) in graphic form*

is a function of the cross-sectional area F (equation 14) and the wetted perimeter P which, under condition cited in Fig. 1, is

$$P = B_1 + 2 \,.\, B_2 \,.\, \sqrt{1 + \frac{1}{m}}\, 2. \tag{20}$$

2. The mean cross-section velocity can be expressed on the basis of equations 2, 19 and 20 in the form

$$V_F = a.\left(\frac{B_2}{m}\right)^b \cdot \left[\frac{\frac{B_1}{B_2} + 1}{\frac{B_1}{B_2} + 2 \,.\, \sqrt{1 + \frac{1}{m^2}}}\right]^b \tag{21}$$

This term for V_F is different from equation 16.

3. The mean velocity V_F^* in the dashed area could not be computed without the assumptions in equation 3. Equation 17, therefore, should be left unaltered.

4. On the basis of this consideration it can be estimated that the required conversion factor (right side of equation 18), or the ratio of velocities V_F / V_F^* respectively, would be given within the limits of inequality

$$\frac{1-k}{1-k^{b+1}} \cdot \frac{\frac{B_1}{B_2} + \frac{2}{b+2}}{\frac{B_1}{B_2} + 1} \geqslant \frac{V_F}{V_F^*} \geqslant \frac{1-k}{1-k^{b+1}} \cdot \left[\frac{\frac{B_1}{B_2} + 1}{\frac{B_1}{B_2} + 2 \,.\, \sqrt{1 + \frac{1}{m^2}}}\right]^b \tag{22}$$

which is a function of four variables. To the three variables already presented in connexion with equation 18, we must add another, namely: the canal bank slope parameter m.

For chosen values of $k = 0.1;\ 0.5;\ 0.9$

$$b = 2/3$$
$$m = 1.5$$

the relation is plotted in Fig. 3. It is evident that the greatest range of inequality limits occurs in relatively narrow canals.

Reduction of cross-sectional area due to weed growth

The area of the cross-section which is not active should be reduced by clearing the banks and bottom. The active part of the cross-section should be measured. It is clear that the channel must be maintained in the cleanest condition for high accuracy in discharge measurement.

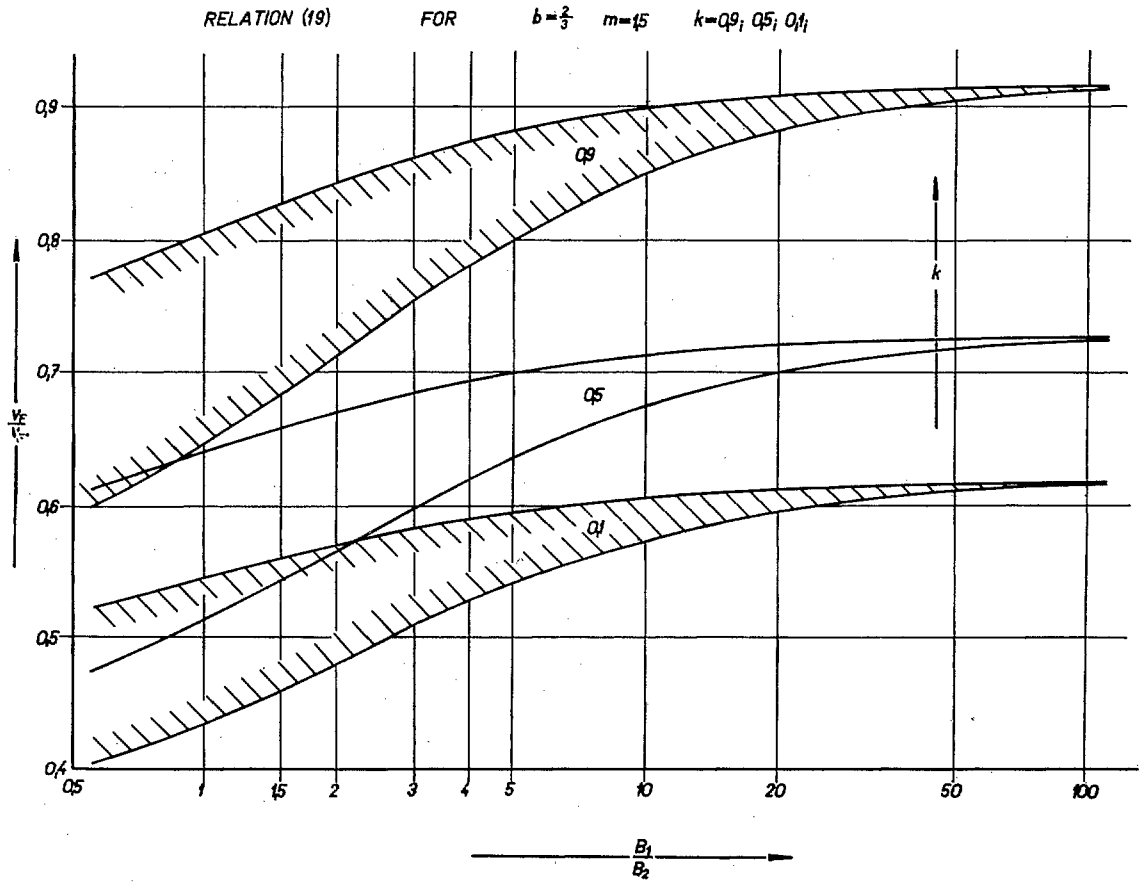


Figure 3. *Graphic expression of inequality (22)*

Comparison of the computed results with measured values

The results of about forty measurements performed with a current meter on the canals of the southern Slovakia lowland were compared. The velocities were also measured by means of floats in the range according to Fig. 1 (Procházka, 1964).

The variable B_1/B_2 was found to lie within the range from 2 to 11, the variable k from 0.1 to 0.7.

When steady, stationary flow was considered, the deviations between the conversion factors measured in the field and those computed according to equation 18 did not exceed the absolute value 0.09 (i.e. a relative deviation of 12 to 15 per cent). It is evident that for a more reliable result it would be necessary to use a larger volume of statistical material.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Procházka, J. 1964. *Štúdia odtokových pomerov odvodňovacích kanálov Žitneho ostrova* [Run-off conditions study on drainage canals of Southern Slovakia]. Bratislava, VÚV.

Les jaugeages au moulinet et au flotteur à l'heure du calcul automatique

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Résumé. L'introduction du calcul automatique en hydrologie a permis de réaliser des progrès spectaculaires ; cependant les résultats reposent sur les mesures de base, en particulier toutes celles conduisant à l'établissement de la courbe de tarage, mesures qui peuvent donner lieu à des erreurs systématiques très importantes. Ces mesures, si classiques soient-elles dans leur principe, conservent tout leur intérêt.

L'amélioration des appareils n'a pas été négligeable ces dernières années, mais elle augmente moins la précision des mesures que l'amélioration des méthodes.

Deux domaines sont particulièrement importants : la mesure des très hautes eaux et celle des très basses eaux.

Dans le premier cas, l'organisation systématique de campagnes de mesures de hautes eaux, l'emploi du cercle hydrographique pour les grands fleuves, de téléphériques avec du matériel parfaitement adapté aux différents cas, d'hélicoptères, l'étude systématique des débits transitant dans les plaines d'inondation, ne doivent pas faire oublier que le moulinet a des limites d'application.

Les jaugeages au flotteur ont également fait des progrès : calcul de la vitesse moyenne superficielle conjugué à des études de conditions hydrauliques, flotteurs utilisables la nuit, flotteur lancés par avion, dispositions spéciales pour évaluer convenablement la section mouillée au moment du maximum.

Les jaugeages par dilution donnent, même en eaux chargées, des résultats très prometteurs. Les procédés indirects permettent souvent de bonnes évaluations à posteriori. Mais il reste encore beaucoup à faire dans le domaine de l'hydrodynamique fluviale où pourraient être obtenues des données précieuses, surtout dans le cas de rivières à fond mobile.

Le second cas est plus mal connu et conduit à des erreurs relatives plus grandes. Citons : l'emploi systématique de dispositifs artificiels et transportables de contrôle, l'aménagement provisoire des sections, les campagnes de jaugeages rapprochées en période de tarissement, l'emploi de micromoulinets.

CURRENT AND FLOAT METERS IN THE COMPUTER AGE

Abstract. Computers have permitted spectacular progress in the hydrological field. Nevertheless, the results of these computations depend on field measurements, particularly the measurements leading to the rating curve which may involve very important systematic errors. Although the principle of this sort of gauging is well known, it is still a subject of great interest.

The apparatuses have been improved over the last few years. However, the improvement in methods has had a greater effect on the precision of measurements.

Two fields are of particular importance – the measurement of very high discharges and the measurement of very low ones.

In the first case, the systematic organization of flood discharge measurements, the use of hydrographic circles for wide rivers, the use of cableways with devices perfectly suited to various situations, the use of helicopters or planes, and the systematic study of discharges in flood plains, must not make us lose sight of the limitations of the current-meter.

Measurements with floats have been improved by the computation of average surface speed in conjunction with studies of hydraulic conditions, floats for night measurement, floats thrown from planes, and special devices for exact estimation of the maximum of floods.

Measurements taken by the dilution procedures give promising results, even in turbid water, and the indirect methods may give, after the flood, passably good assessments. But much remains

to be done in the field of hydrodynamic conditions of the rivers where research may lead to interesting data, especially for rivers with unstable beds.

The second case, low flow measurements, is not so well known and may give bigger relative errors. These may be reduced by various procedures: the systematic use of artificial and portable devices for the measurement of flow, the temporary setting-up of measuring sections, repeated gauging during the recession period, and the use of micro-current meters.

MEDIDAS CON FLOTADORES UTILIZANDO CÁLCULO AUTOMÁTICO

Resumen. Los ordenadores han permitido un progreso espectacular en el campo de la hidrología ; sin embargo, los resultados de estos cálculos se apoyan en mediciones de campo, especialmente en las mediciones de la curva de calibración. Estas mediciones pueden incluir errores sistemáticos de considerable importancia. Este tipo de aforo es bien conocido, al menos en principio, pero mantiene, incluso en el presente, un gran interés.

La mejora de los aparatos no ha sido despreciable en el transcurso de los últimos años, pero su efecto en la precisión de las medidas no es tan importante como la mejora de los métodos. Hay dos campos que son de particular importancia: la medición de caudales muy elevados y la de caudales muy bajos.

En el primer caso, el establecimiento sistemático de mediciones de caudales de crecidas, el uso de círculos hidrográficos para ríos anchos, el empleo de pasarelas de cable con dispositivos perfectamente fijados a las diversas situaciones, el empleo de helicópteros o aviones, el estudio sistemático de caudales en planos de inundación, han hecho posible algunos progresos, pero el medidor de corriente presenta bastantes limitaciones.

Las mediciones hechas con flotadores se han mejorado en el cálculo de la velocidad superficial media conjuntamente con los estudios de condiciones hidráulicas, flotadores para medición nocturna, flotadores arrojados desde aviones, y dispositivos especiales para una buena estimación de la sección transversal para los máximos de crecidas.

Las mediciones por los procedimientos de dilución dan, incluso con aguas turbias, resultados prometedores, mientras que los métodos indirectos pueden dar después de la inundación información bastante aceptable. Pero queda mucho por hacer en el campo de las condiciones hidrodinámicas de los ríos, principalmente en los de cauce inestable, en los que las investigaciones han llegado a resultados interesantes.

El segundo caso, mediciones de caudal reducido, no es tan conocido y puede dar errores relativos mayores. Estos se pueden reducir por diversos procedimientos: el empleo sistemático de dispositivos artificiales y con frecuencia desplazables para el control de la corriente, disposiciones provisionales en el cauce de pequeños ríos, el ajuste de calibración repetido en el tiempo a lo largo de periodo de recesión, y el uso del micromedidor de corriente.

ИЗМЕРЕНИЕ РАСХОДОВ ВОДЫ ВЕРТУШКАМИ И ПОПЛАВКАМИ ПРИ АВТОМАТИЧЕСКИХ ВЫЧИСЛЕНИЯХ

Резюме. Вычислительные машины обеспечили определенный прогресс в области гидрологии. Тем не менее результаты этих вычислений основываются на полевых измерениях, а именно, на измерениях, обеспечивающих получение кривой расхода, которая может включать весьма значительные систематические ошибки. Принцип этих измерений хорошо известен и в то же время представляет большой интерес.

За последние несколько лет приборы систематически улучшались. Однако улучшение методов обеспечивало больший эффект для точности измерений.

Две области особенно важны : измерение весьма высоких и весьма низких расходов. В первом случае некоторый прогресс стал возможен благодаря систематическим измерениям паводочных расходов, использованию гидрографических кругов для широких рек, использованию тросовых переправ с устройствами, пригодными для различных условий, использованию вертолетов и самолетов и систематическому изучению расходов в поймах, но вертушки имеют свои ограничения.

Измерения с помощью поплавков были улучшены благодаря вычисления средней поверхностной скорости в сочетании с изучением гидравлических

условий, применением поплавков для ночных измерений, поплавков, сбрасываемых с самолетов, и специальных приспособлений для точной оценки максимальных расходов паводков.

Измерения методом смешения дают обнадеживающие результаты даже в мутной воде; косвенные методы, применяемые после прохождения паводка, могут быть оценены как хорошие. Но многое остается сделать в области изучения гидродинамических условий рек, что может привести к интересным данным, главным образом, для рек с неустойчивым руслом.

Во втором случае измерение малых расходов до сих пор недостаточно хорошо изучено и может вызвать еще большие относительные ошибки. Они могут быть уменьшены различными способами: систематическим использованием искусственных, а в некоторых случаях, съемных устройств для регулирования стока, предварительным улучшением русел малых рек, повторным измерением расхода в период спада и использованием микровертушек.

Lorsqu'on fait un parallèle entre les moyens de calcul automatique employés actuellement pour le traitement des données hydrologiques, d'une part, et les procédés d'observations et de mesures, d'autre part, on est souvent frappé du caractère un peu primitif de ces derniers, et la tentation est grande de reporter tous ses efforts sur des programmes astucieux de traitement de données ou de méthodes d'interprétation élégante en négligeant cette chère vieille chose : la mesure, opération bien connue et pour laquelle il n'y a pas beaucoup d'amélioration à espérer.

Pendant des décennies, on s'est satisfait d'une précision de 10 % sur la mesure des débits, même lorsque cette mesure était facile. Autrefois, en effet, la faible taille des échantillons statistiques aurait rendu inutile une plus grande précision. Mais les utilisateurs de données sont maintenant, à juste titre, beaucoup plus exigeants et on peut effectivement (et on doit faire) beaucoup mieux avec des échantillons beaucoup plus grands et des procédés de mesures améliorés. Or l'ordinateur ne peut sortir de résultats qu'en fonction de la qualité des données qui y ont été introduites ; le meilleur modèle de simulation conduira, dans la pratique, à des résultats catastrophiques si les données sont entachées d'une erreur *systématique* de 20 ou 30 %, ce qui, malheureusement, n'est pas si rare qu'on le croit. Or l'établissement de la courbe de transformation hauteur-débit peut s'avérer lourd de conséquences, car, contrairement aux fantaisies de lecture ou d'enregistrement des hauteurs d'eau, les erreurs dans la courbe de jaugeage conduisent à des écarts systématiques souvent plus difficiles à détecter à l'ordinateur que par les anciennes méthodes manuelles. L'intérêt de disposer de bonnes mesures est donc encore plus grand qu'autrefois, en raison de la précision plus grande qu'on exige.

Que peut-on faire pour améliorer ces courbes de tarage ?

Tout d'abord sont-elles stables et univoques ? Si elles ne le sont pas, il ne s'agit pas d'un problème d'appareil ou de procédé de mesure : c'est une question d'implantation de stations et d'exploitation de réseaux.

Il faut, en cas de lit instable, ou chercher une section stable, même si l'écoulement y est particulièrement anarchique, et séparer alors résolument la station de mesure de hauteurs de celle de la mesure des débits, ou créer une station artificielle stable, ce qui est extrêmement coûteux et pas toujours efficace. Si cela n'est pas possible, il faut multiplier les jaugeages et les combiner avec des mesures de pente, ce qui conduit à concentrer les efforts sur un petit nombre de stations bien choisies.

Si la station n'est pas univoque, la méthode bien connue des deux échelles, combinée avec un grand nombre de jaugeages, permet d'établir puis d'utiliser un

réseau de courbes de transformation en fonction de la hauteur et de la pente. Retenons de tout ceci qu'il serait utile, pour de nombreux services hydrologiques, de donner des instructions pour un dépistage systématique des stations à lit instable ou à courbes de tarage non univoques.

Si la courbe est univoque, avec les moulinets actuels, il est possible, pour les basses eaux et les eaux moyennes, d'établir la courbe de transformation hauteur-débit à 2 % dans les nombreux cas où l'écoulement est uniforme, à condition bien entendu d'effectuer les mesures sur un nombre de verticales et en un nombre de points suffisants et d'employer un mode de dépouillement qui ne donne pas lieu à des erreurs systématiques supplémentaires, ce qui est trop souvent le cas.

Par contre, les deux extrémités de la courbe de tarage correspondant aux crues et aux très basses eaux sont presque toujours établies avec beaucoup moins de précision. Qu'a-t-on fait jusqu'ici dans notre service pour améliorer cette situation ?

1. JAUGEAGES DE CRUE

Admettons, en première approximation, que le moulinet soit le seul moyen de mesure. Avec cet appareil nous avons pu, dans bon nombre de pays africains, obtenir des courbes d'étalonnage très valables jusqu'à des crues dont la fréquence est décennale et parfois même centenaire. Mais dès que les vitesses dépassent 2 m/s, nous avons rencontré de sérieuses difficultés. Dans ce cas, ce n'est pas seulement le perfectionnement de l'appareil lui-même qui résoudra les problèmes de crue. Tout ce que l'on peut faire, et les constructeurs s'y sont employés, c'est d'éviter qu'il soit nécessaire de le nettoyer trop souvent lorsque les eaux sont chargées, et faire en sorte que sa mise en œuvre soit rapide.

En fait, le plus difficile en période de crue, sur des rivières rapides, est d'amener et de maintenir le moulinet dans la section de mesure et plus précisément sur sa verticale, avec des vitesses élevées et pas mal d'épaves.

Les ponts peuvent paraître bien commodes, mais lorsqu'ils ont plus d'une arche l'écoulement mauvais ou très mauvais ne permet guère d'espérer une bonne précision ; en cas de crues très violentes comme celles de 1969 en Tunisie, il est préférable de ne pas y stationner pour ne pas être emporté avec elles, enfin, même dans des cas plus favorables, la section sous le pont peut être franchement mauvaise pour diverses raisons. Il y a aussi bien des cas où il n'y a pas de pont.

On peut alors jauger d'une embarcation : nous utilisons pour cela, depuis plus de dix ans, des canots pneumatiques "Zodiac" pourvus de moteurs hors-bord, qui peuvent admettre des charges utiles allant jusqu'à 1 500 kg et pour lesquels on peut garantir au moins trois ans de service dans les dures conditions de l'Afrique tropicale. Ces bateaux sont amarrés sur le câble par dispositif de largage semi-automatique. Pour éviter tout sabotage, et pour des raisons d'économie, les câbles sont installés à chaque mesure : le câble est enroulé sur un petit treuil de 500 kg fixé solidement sur une berge, ce treuil permet la mise en tension facile du câble une fois que celui-ci a été traversé et amarré sur la berge opposée. Mais si les débris flottants sont nombreux et de grande taille, la traversée du câble devient très difficile dès que la largeur dépasse 400 m, pour des vitesses inférieures à 2 m/s, ou 100 m pour des vitesses de 3 à 4 m/s. Un câble fixe peut être mis en place en saison sèche, mais s'il n'est pas de gros diamètre, on risque de ne plus le retrouver en arrivant à la station : des câbles de 4 mm tentent beaucoup les chasseurs africains.

On ne peut pas utiliser de câble et repérer la position du bateau une fois à l'ancre, par visées au cercle hydrographique à partir de balises sur les rives. C'est ce que nous faisons fréquemment sur les grands cours d'eau congolais ou de la République centrafricaine. Mais, dans ce dernier cas, sauf pour de grands fleuves tels que le Congo, des bateaux aussi stables que le canot pneumatique ne peuvent plus être employés pour des vitesses supérieures ou égales à 4 m/s, en raison des débris flottants qui s'accumulent sur l'équipement hydrométrique et la chaîne d'ancre et des risques de chocs avec les plus gros d'entre ces débris.

Ajoutons que nous avons atteint ces vitesses avec des hydrologues très expérimentés, toute une organisation pour repérer les épaves avant qu'elles n'arrivent sur le bateau et en maintenant le moteur en marche pour étaler exactement le courant et pouvoir s'échapper rapidement en cas de danger. Il importe d'ailleurs que ces évolutions dangereuses ne durent pas trop longtemps et les appareils de jaugeage par intégration, en laissant descendre régulièrement le moulinet sur la verticale, font gagner du temps. Ces vitesses de 4 m/s ne sont pas dépassées, heureusement, pour la plupart des cours d'eau calmes. Mais ce n'est pas le cas des oueds nord-africains et des rivières à crues cycloniques.

Il n'est plus possible alors de s'aventurer en bateau sur la rivière et l'on en arrive à la solution de la station téléphérique, solution coûteuse dès que la largeur dépasse une centaine de mètres. Evidemment, il y a la solution que nous utilisons en Nouvelle-Calédonie : pour une station, nous installons un téléphérique et un baraquement démontable bien ancré au sol : dès l'annonce du cyclone, les hydrologues rejoignent leur baraquement et après le cyclone, sauf accident, la rivière est jaugée pour les hautes eaux ; on démonte le tout et on le remonte à la station suivante pour le prochain cyclone. On a fait cela de façon moins régulière à Madagascar. Mais de toutes façons, les mesures au moulinet deviennent généralement impossibles pour des vitesses supérieures ou égales à 5 m/s, les épaves devenant trop nombreuses.

Il y a encore la solution du jaugeage par hélicoptère, mais celui-ci est difficile, peu précis, et ne se conçoit guère que pour la mesure d'un grand fleuve. Notons qu'en période d'inondation, il est difficile de trouver un hélicoptère disponible. Ils sont réservés pour l'évacuation des sinistrés. De toute façon, avec des vitesses supérieures à 5 m/s, il faut abandonner le moulinet.

Mais, auparavant, jetons un dernier coup d'œil sur une cause d'imprécision dans les jaugeages de crues : l'écoulement dans les plaines d'inondation. Le secret d'une bonne mesure, dans ces plaines, suppose une parfaite connaissance de l'écoulement ; le choix d'une section avec écoulement régulier et pas trop lent est capital. Signalons à ce sujet les services que peuvent rendre les routes submergées, en légère surélévation, perpendiculaires si possible à la direction du courant. Bien souvent, les vitesses faibles ou les faibles hauteurs de submersion imposent le micromoulinet ou même le jaugeage par flotteur immergé. Le plus simple d'ailleurs est de choisir, quand on le peut, une section sans plaine d'inondation, même si elle est située à une bonne distance de l'échelle ou de l'enregistreur.

Pour en terminer avec les jaugeages au moulinet, précisons qu'autant que possible, on doit commencer le jaugeage après le maximum de la crue, les débits en surface deviennent moins nombreux et l'immersion du moulinet présente ainsi moins de risques.

Les moyens indiqués ci-dessus, correctement mis en œuvre par des opérateurs expérimentés, permettent la mesure de crues très importantes dans de bonnes conditions, pour un grand nombre de régions du monde. Le plus souvent, lorsqu'on manque de mesures de crues, ce n'est pas parce qu'il n'était pas possible de les faire,

mais parce qu'elles n'ont pas été faites tout simplement. Alors, on prend l'habitude d'extrapoler, de pallier le manque de mesures par des corrélations plus ou moins valables et l'on établit ainsi, tant bien que mal, les projets. Contrairement à ce qu'on pourrait croire, cette habitude est plus répandue dans les pays hautement évolués que dans les pays en voie de développement.

Les jaugeages par dilution apporteront une solution pour bien des cas : ceux-ci peuvent en effet être utilisés en forte crue : à Madagascar, les hydrologues de la Division technique générale ont, avec la collaboration de leurs collègues de l'ORSTOM, mesuré par cette méthode 2 200 m^3/s (avec reconcentration), mais pour les forts débits et les eaux chargées, les opérateurs doivent être très expérimentés avec une très bonne connaissance de la rivière à jauger. Avant une campagne de jaugeages de crue par dilution, il faut disposer d'un stock suffisant de bichromate de sodium et d'une équipe d'hydrologues ayant une parfaite maîtrise de ce type de mesure, ce qui est rarement le cas. Cependant, dans bien des pays à crues violentes, il y aurait intérêt à examiner la possibilité d'employer une telle méthode.

Très fréquemment, on en revient à la mesure de jaugeages au flotteur, mais ceux-ci exigent beaucoup plus de précautions qu'on ne le croit : il faut d'abord un écoulement relativement régulier sur une longueur suffisante : 10 fois ou 5 fois la largeur de la rivière. Mais dans le cas de rivières assez larges, cette condition est très difficile à réaliser. On se contente parfois d'une base de 100 m correspondant à 20 secondes pour 5 m/s. Nous cherchons à établir la courbe des vitesses superficielles sur toute la largeur du cours d'eau, ce qui exige un système de repérage, plusieurs opérateurs et un très bon système de signalisation entre ces opérateurs. Là aussi, il vaut mieux ne pas improviser. Nous avons utilisé des flotteurs artificiels : bouteilles de bière ou cubes de bois avec une pile et une ampoule électrique pour être utilisés la nuit ; mais le plus souvent, pour les très fortes crues, nos hydrologues ont tiré parti des flotteurs naturels : ce sont des arbres en général. Cela a été le cas pour les dernières crues de Tunisie et de Madagascar. Sur une seule station de jaugeage de Tunisie, les hydrologues de BIRH et de l'ORSTOM en ont utilisé 900 pour une seule pointe de crue (oued Zeroud : 18 000 m^3/s). La déformation régulière des courbes de vitesses superficielles avec la hauteur d'eau à l'échelle montre bien la faible dispersion des vitesses ainsi estimées.

Le professeur Ouryvaev avait indiqué au colloque de Leningrad un procédé de mesure de vitesse en avion par largage de produits colorants dans la section et deux photographies aériennes à court intervalle de temps dans la section permettant de déterminer les temps parcourus et par la suite les vitesses pour les divers points de largage.

Le rapport entre vitesse superficielle moyenne et vitesse moyenne dans la section est estimé, d'après les jaugeages réguliers, correspondant aux plus forts débits mesurés. Si on sépare bien, pour cette estimation, les plaines d'inondation ou les zones à faible profondeur du chenal principal, on aboutit à des coefficients stables dont la valeur est comprise généralement entre 0,90 et 0,95 pour des profondeurs supérieures à 3 ou 4 mètres.

Il reste à régler le problème le plus difficile : celui de l'estimation de la section mouillée, c'est-à-dire du fond. Sur l'oued Zeroud, le problème était simple, le lit a été décapé jusqu'au rocher au moment du maximum. Cela a été vérifié par sondage au saumon de 100 kg dépouillé à cet effet de la pédale du moulinet et de la queue, alors que les jaugeages au moulinet étaient devenus impossibles pour les débits déjà bien inférieurs. Ailleurs, on a essayé d'utiliser le procédé indiqué par L. Leopold pour les études géomorphologiques : enfouir dans le lit des chaînes verticales ancrées à un

corps mort, sur tout le profil en travers. On peut aussi utiliser des empilages de briques, les briques supérieures étant emportées par la crue. Ces opérations sont délicates, mais elles sont préférables à celles qui consistent à relever le profil en travers avant et après la crue, car, au moment du débit maximal, la cote absolue minimale du profil en travers est souvent plus faible qu'au début ou à la fin de la crue. Là encore, on obtient le maximum de précision avec un lit stable. On tirerait des avantages certains d'études d'hydrodynamique conduisant à des données quantitatives sur les déformations du lit des fleuves.

En comparant les résultats des jaugeages au flotteur, les données des formules d'écoulement, les courbes extrapolées des vitesses moyennes à partir des jaugeages réguliers, nous arrivons à des estimations pas trop imprécises des crues exceptionnelles, mais cela exige que l'on considère ces mesures au flotteur comme des opérations difficiles pour lesquelles de grandes précautions sont à prendre et non comme un vague dépannage à résultats plus ou moins qualitatifs. Encore faut-il être là quand arrive la crue. Aussi, bien souvent, doit-on employer un mode d'estimation indirect : à posteriori. Nous ne dirons rien à ce sujet, il existe une note technique très complète de l'OMM 225 T.P. 119 : "Measurement of peak discharge by indirect methods".

Ce sont alors les données ainsi établies qu'on introduit dans l'ordinateur pour l'étude de la distribution des crues. On voit, dans le cas où les jaugeages réguliers ne sont déjà plus possibles pour le maximum annuel, qu'il est facile, malgré de grandes précautions, de faire une erreur systématique de 15 à 20 % et sans précautions de faire des erreurs atteignant et dépassant 100 %.

2. VALEURS MINIMALES DES DEBITS

Contrairement à ce que pensent bien des ingénieurs de génie hydraulique, la connaissance des valeurs minimales des débits est encore plus mauvaise que celle des crues exceptionnelles pour trois raisons : (a) les valeurs très rares laissent des traces moins nettes que les crues exceptionnelles pour la bonne raison que les niveaux atteints sont vite submergés ; (b) dans le cas de vitesses très faibles, l'irrégularité spatiale est très grande dans un même profil en travers ; (c) il ne peut pas exister de règle pour l'extrapolation vers le bas des courbes hauteur-débit.

En ce qui concerne les appareils, signalons les services rendus par les micro-moulinets, les petits déversoirs transportables et même les flotteurs lestés et immergés pour les très faibles vitesses.

En région tropicale où les débits de basses eaux sont souvent très faibles, nous procédons souvent, avant le jaugeage, à quelques travaux d'aménagement de l'écoulement, de façon à opérer dans un petit canal à écoulement aussi régulier que possible. Le choix d'une bonne section de basses eaux est essentiel.

Mais, pour les crues comme pour les étiages, ce qui constitue un progrès énorme c'est de prévoir, dans l'organisation d'un service, des campagnes de mesures régulières de basses eaux tous les ans, avec plus de précision et des mesures en un plus grand nombre de points si l'étiage est exceptionnel ; on évitera alors parfois, au passage de l'ordinateur, des erreurs de 1 000 % (mille), telles que celles que nous avons déjà vues. Cela se produit si le débit de la rivière devient très faible en basses eaux et s'il n'y a pas tous les ans des jaugeages d'étiage. Cela sera de nature à faire réfléchir les spécialistes de l'alimentation en eau des agglomérations.

En conclusion, il importe de produire, dans le cadre de l'exploitation des réseaux, des données dont la précision soit en rapport avec la puissance des moyens de calcul qui les utilisent. Pour cela on ne doit pas se croiser les bras en invoquant le caractère primitif des moyens dont on dispose pour l'estimation des valeurs maximales et minimales des débits. Il convient de prendre le problème de mesure au sérieux et de ne pas se reposer, les yeux fermés, sur les techniciens qu'on abandonne à leur triste sort sur le terrain. Il faut mener intelligemment ces mesures qui requièrent beaucoup plus de compétence, de jugement et d'imagination qu'on ne le pense. Enfin, il ne faut pas hésiter à mettre au programme de l'exploitation des réseaux, des campagnes systématiques de jaugeages d'étiage et de crues exceptionnelles en prévoyant bien à l'avance les moyens nécessaires.

DISCUSSION

Acheampong: I would like to know M. Rodier's experience in measuring accurately the discharge of floods in Africa. Our problem in Ghana is that high flows may overflow the banks of a river even before reaching the peak so that a portion of the flow does not pass through the measuring section. I should like to know whether M. Rodier has taken this into account during his measurement of flood discharges in Africa.

Rodier: On vient de parler des mesures dans les plaines d'inondation, qui sont effectivement quelquefois oubliées. J'en parle dans l'article en question. Nous avons systématiquement procédé à ces mesures. Je citerai deux chiffres : sur le Sénégal, par exemple, pour la crue de période de retour de 10 ans, le débit passant dans la plaine d'inondation est égal au tiers du débit total. Sur une autre rivière, le Logone, pour un débit de l'ordre de 3 000 m^3/s pour la crue décennale, le débit passant dans la plaine d'inondation était le quart du débit total. En Afrique, on ne peut évidemment pas faire autrement que de mesurer les débits dans les plaines d'inondation, sinon on commet de très graves erreurs. J'en parlerai davantage au représentant du Ghana s'il le désire, car ce serait trop long de faire un exposé sur les méthodes employées dans les plaines d'inondation.

Measurement of discharge under ice cover

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Abstract. Many difficulties are involved in the measurement of discharge under ice cover, particularly in the operation of hydrologic data-gathering networks located in Arctic and Sub-Arctic regions. Difficulties result from extreme ice thickness, double thicknesses of ice, large depths of slush ice, coincidence of spring flows with ice break-up, freeze-up of gauging equipment in sub-zero temperatures, etc. Some of these problems are discussed and procedures used in the operation of networks in northern areas of Canada are outlined.

A possible technique for estimation of river discharge using single velocity in a cross-section is briefly described and comparison in accuracy is demonstrated between discharges derived using this technique and measured discharges using standard methods.

MESURE DU DEBIT SOUS UN CHAMP DE GLACE

Résumé. De nombreux problèmes sont liés à la mesure du débit sous un champ de glace, notamment lorsqu'il s'agit de faire fonctionner des réseaux de collecte de données hydrologiques situés dans les régions arctiques et subarctiques. Les difficultés rencontrées ont eu pour origine l'extrême épaisseur ou la double épaisseur de la glace, un sorbet très épais, la coïncidence du débit de printemps et de la débâcle, la congélation de l'équipement de mesure par des températures négatives, etc. Ces problèmes sont passés en revue dans cette communication dans laquelle sont également exposées, dans leurs grandes lignes, les techniques utilisées pour l'exploitation des réseaux dans les régions septentrionales du Canada. On y trouve une brève description d'une technique permettant d'évaluer le débit des cours d'eau au moyen d'une seule vitesse dans une section transversale, ainsi que des comparaisons entre les précisions atteintes pour les débits obtenus grâce à cette technique et les débits mesurés à l'aide des méthodes courantes.

MEDICIÓN DE CAUDALES BAJO CAPAS DE HIELO

Resumen. Se encuentran numerosas dificultades para la medición de caudales bajo capas de hielo, particularmente cuando se trata del funcionamiento de las redes hidrológicas situadas en las regiones árticas y subárticas. Dichas dificultades son debidas al espesor del hielo, a dobles capas de hielo, a grandes profundidades de hielo pastoso, coincidencia de los caudales de primavera y del deshielo, congelación del equipo de medida con temperaturas bajo cero, etc. Se discuten algunos de estos problemas y se describen los procedimientos utilizados en el funcionamiento de las redes en las áreas del norte del Canadá.

Se describe con brevedad una técnica posible para la estimación del caudal de los ríos utilizando una velocidad única en una sección transversal y la comparación en precisión con los caudales obtenidos utilizando esta técnica y los caudales medidos utilizando los métodos convencionales.

ИЗМЕРЕНИЕ РАСХОДА ПОД ЛЕДЯНЫМ ПОКРОВОМ

Резюме. Много трудностей встречается при измерении расхода под ледяным покровом, особенно при сборе гидрологических данных на сети, находящейся в арктических и субарктических районах. Эти трудности вызваны урезмерной толщиной льда, двойной толщиной льда, толстым слоем шуги, совпадением весенних расходов со вскрытьем реки, обмерзанием измерительной аппаратуры при температурах ниже нуля и т. д. В докладе рассмот-

рены некоторые из этих вопросов и освещены вопросы методики проведения сетевых работ в северных районах Канады.

Кратко описана возможная методика оценок речного расхода, используя единичное измерение скорости в попречном сечении, и показано сравнение точности между расходами, полученными этии способом и расходами, измеренными стандартными методами.

NAVIGATION OF AIRCRAFT

Travel for the purpose of streamflow data collection in northern Canada is usually via single-engine aircraft, such as the De Havilland Beaver or Otter. Common Arctic winter conditions, such as snow cover, drifting snow and ice-fog, obscure river channels, lakes and other natural landmarks making map-reading for navigational purposes extremely difficult. The sparsely located settlements of the Canadian north are all equipped with radio-beacons which are invaluable when flying from settlement to settlement, but they are of very little assistance when flying to a gauging site. It takes a great deal of concentrated effort under prevailing lighting conditions to locate a gauging station in these circumstances. Aircraft travel also becomes extremely frustrating when, for example, hours are spent getting ready to fly only to find a grey blanket of ice-fog settling in around the aircraft. These situations are frequent and successful arrival at a station is a major accompishment. To assist navigation under the conditions described, gauging stations in the Arctic regions normally are equipped with marker pylons which can be described as steel tripods, approximately three metres in height with large triangular plates mounted at the apex and painted fluorescent orange (Fig. 1). Where possible, these markers are located on a prominent point of land. It has been found that the pylons greatly facilitate location of stations from the air under conditions of complete snow cover.

ICE THICKNESS

In Arctic and Sub-Arctic regions, ice thicknesses in the order of three metres or more combined with large depths of slush are not uncommon. It is possible to mitigate or circumvent the complicating factors associated with difficult ice cover of this nature by judicious selection of winter measurement sites. It is a rather common occurrence for example, to find reaches of open water immediately below the outlet of large lakes. The erection of a cable way at such a location reduces the problems of winter measurements (Fig. 2). Likewise, a satisfactory winter measurement section frequently can be located at the lip of a rapids where thin ice occurs, thus alleviating ice-cutting problems. In addition, backwater effect can be substantially reduced or eliminated entirely by locating the gauging station above a rapids or a waterfall. For example, at the Back River at latitude 67° 40' north, backwater effect has not been known to exceed 35 centimetres, although ice thickness at the gauge site exceeds 3 metres during most winters. The rapids, which are the control for the station, remain free of ice with the exception of the water's edge (Fig. 3).

ICE CUTTING

Where it is not possible to avoid cross-sections with large depths of ice, ice cutting becomes the major obstacle to the proper conduct of winter discharge measurement



Figure 1. *A marker pylon*



Figure 2. *A winter measurement site*

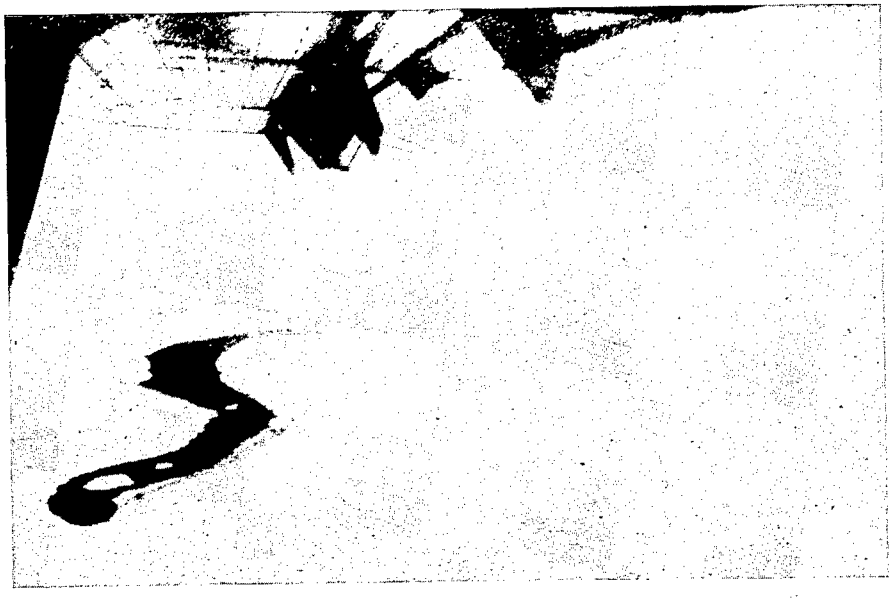


Figure 3. *The Back river*



Figure 4. *A powered ice drill*

programmes. In Canada, a powered ice drill is now a common tool for this purpose (Fig. 4). Its power pack, weighing approximately 9 kg, consists of a 3 horsepower, air-cooled, two-cycle engine such as is used in North America on lawn mowers or snow blowers, an appropriate set of reduction gears, and a clutching assembly. Fastened to the power pack by means of a splined connector is a flighted auger shaft and toothed cutting head, the shaft being of appropriate length to penetrate the ice layer. In the District of Keewatin, where ice thickness averages 2.8 metres in late winter, it is common practice to drill the necessary holes in two stages; a 1.5 metre length of auger shaft is used first, followed by a second shaft of greater length (Fig. 5).

A multi-purpose tool is used to measure ice thickness where only a simple ice cover exists (Fig. 6). The graduated shaft of this measuring device is fabricated out of wood or metal and has attached at its lower end two perforated semi-circular plates which fold upwards from a normal horizontal position. When this tool is thrust downwards through the ice holes, the plates fold to allow easy passage. When it is raised, the plates return to a horizontal position, thus permitting the detection of the underside of the ice for the purpose of determining ice thickness. When the tool is moved upward through the drilled hole, ice chips and fragments resulting from drilling are easily removed.

OBSERVATION OF WATER DEPTH AND VELOCITIES

Co-incident with the increased use of the powered ice drill, a need arose to redesign standard metering equipment in order that it might be accommodated in a drilled hole of approximately 20 centimetres in diameter. Two weight assemblies with a maximum diameter of 16 cm were developed. The "Calgary" type weighs 18 kg and employs a tear-drop shaped lead weight. The "Winnipeg" type weighs 8.2 kg and also employs a tear-drop shaped weight with some modification for lower resistance to the current (Fig. 7). Both types have proven to be fairly stable in velocities under 2 metres per second. Incorporated within the framework for the weight is a modified pattern 622 Price-type current meter. The entire assembly fits easily through the drilled hole.

A sounding reel or a handline is used to suspend the weight assembly where depth of water is greater than 3 or 4 metres. The reel is mounted on a collapsible support set on runners (Fig. 8). Because of the extremely cold weather, the support is equipped with a chamber heated by catalytic heaters to keep the meter from freezing while moving the equipment from one position to the next. Where depth of water is less than 4 metres, graduated steel rod is used.

SLUSH ICE CONDITIONS

The problems presented by extreme ice thickness are aggravated further by the formation, during freeze-up, of thick slush ice layers which adhere to the under-surface of the ice cover. Slush ice is formed on the surface of moving water when it is exposed to sub-zero air temperatures. This slush ice, in the form of ice panes, slivers and crystals, is swept under newly formed ice-blankets and is trapped there. Heaviest slush ice-formations occur in the early winter and then gradually dissipate or form a part of the solid ice cover. Where the combined thickness of ice and slush does not



Figure 5. *Ice cutting*

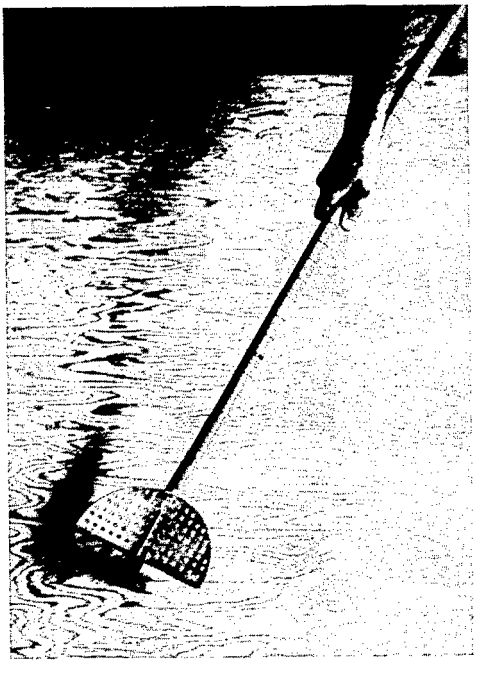


Figure 6. *Device for measuring ice thickness*



Figure 7. *The "Winnipeg" weight assembly*



Figure 8. *Sounding reel mounted on runners*

exceed 6 metres, a sectional, flanged slush pole is used initially to loosen the slush formations sufficiently to allow easy passage of the weight and meter assembly. Where the combined depths are greater, a specially designed elliptical weight is used to penetrate the slush horizon (Fig. 9). This weight, which has enlarged tail fins, is suspended at a point slightly off the centre of gravity, permitting it to assume a vertical, nose-up position when suspended in the air. When the weight is lowered through the slush ice in this position, the enlarged tail fins act as cutting and breaking edges. When lowered below the interface between water and slush, a slight current is sufficient to force the weight into a horizontal position. Accurate determination of the interface is obtained by raising the metering assembly until the meter rotor is stopped. The distance from the water surface to this point is then noted as ice thickness.

RECORDING OF WATER LEVELS

Low temperatures, permafrost and rocky terrain preclude the use of conventional float-operated water stage recorders. Two types of pressure water-level recorders are being used exclusively to record stages at isolated locations in northern Canada, both of which utilize nitrogen bubble systems to transfer river stage to the recording pen. One recorder is the mercury manometer gauge which balances a column of mercury against the pressure due to river stage (Fig. 10). The other recorder is the "Winnipeg" type pressure gauge in which the pressure due to river stage is transferred to the recording pen by means of mechanical linkage (Fig. 11). To operate at prevalent temperatures, it is necessary to provide a heated shelter for the mercury manometer gauge in order to maintain the temperature of mercury above its freezing point. The "Winnipeg" type gauge does not require heat for its operation. However, the Stevens strip chart drive, common to both these instruments, is unreliable at temperatures of about -40°C, consequently a variety of propane-fuelled heaters are in use in northern Canada. Experimentation with electrical heat, provided by a wind-driven generator (Fig. 12), was carried out in the Northwest Territories but fuelled heaters are found to be more reliable and economical. Experimentation also has been made with special low-temperature lubricants such as "Moebius Syntalube", which indicates that reliability of the Stevens strip chart recorder can be improved to temperatures of -45°C.

ALTERNATIVE STREAMFLOW MEASUREMENT TECHNIQUE

Standard procedure in the making of a current meter discharge measurement requires that the total area of the cross-section at the place of measurement be divided into small or partial sections, and the area and the mean velocity of each be determined separately. The sum of the discharges in all the partial sections is the discharge of the stream.

This procedure, if correctly applied, is the most accurate of the known methods of measuring the discharge of a stream. However, this procedure is time-consuming, and what follows is a demonstration of one possible alternative.

This alternative is based on the belief that a relationship exists between discharge computed on the basis of single velocity in a cross-section and that obtained using the above method in the same cross-section. In pursuit of this possibility, a limited

analysis was made of four gauging stations located at points ranging from the United States - Canada border to near the Arctic circle. This analysis indicates that such is the case. If further study confirms this relationship, then this time-saving technique in taking discharge measurements could be introduced.

METHOD OF ANALYSIS

Results of discharge measurements for several years were listed in Table 1 in columns (1) to (4) for:

Seal river below Great Island
Location—Latitude 58° 53′, Longitude 96° 16′, Man.
Drainage area—48,200 km^2.
Gauge—Recording
Operation—Continuous
Maximum discharge—1,300 m^3/sec
Minimum discharge—70.4 m^3/sec

Each discharge measurement was examined, and maximum velocity in the cross-section at 0.2 depth was listed in column (5). Where possible the same vertical was used whether maximum velocity occurred each time at that vertical or not. However, since verticals used for each measurement are not always coincident, a deviation of ± 3 metres was allowed in the selection of the vertical.

The velocities listed in column (5) were then multiplied by the area for the measurement (column 3) and results listed in column (6). The discharge figures in column (6) were then plotted against measured discharge listed in column (4), and correction factors computed (Fig. 13).

Discharges on the basis of single velocity in the cross-section were then arrived at by multiplying the values in column (6) by the correction factor and the results, together with the percentage deviation from measured discharge, listed in the last two columns of each table. These discharges were then plotted against gauge height and maximum deviation from the curve computed.

DISCUSSION

From the four gauging stations studied it appears that:
(a) Estimate of discharge to approximately ± 10 per cent accuracy can be computed using the formula

$$Q = A \times 0{,}70\ V_{max}$$

where Q = discharge in cubic metres per second; V_{max} = Velocity at 0.2 depth at maximum velocity point within the cross-section; 0.70 = Correction factor.

(b) The correction factor (0.70) appears to be constant regardless of the latitude of gauging stations studied, with the possible exception of Seal river, where the average factor appears to be 0.65. It is not essential that the correction factor be constant, although this is of considerable interest.

The above relationship was developed using velocities as observed at 0.2 depth at the maximum velocity point within the river cross-section. Since velocity distribution in the vertical is normally a parabola it is possible that a similar relationship exists using velocity observed at constant depth, say 1 metre above streambed.



Figure 9. *Elliptical weight designed to penetrate the slush horizon*

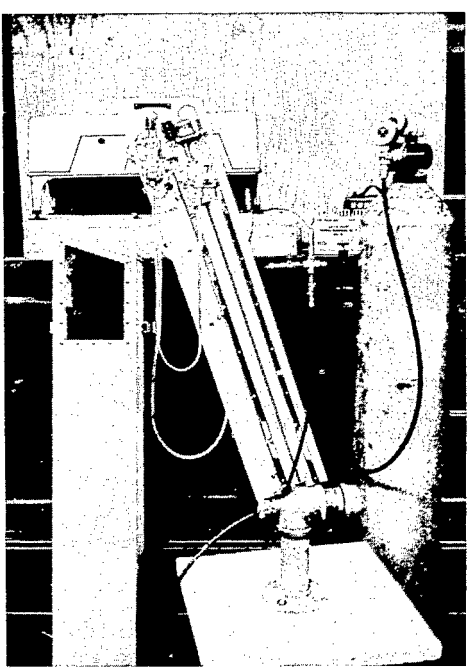


Figure 10. *Mercury manometer gauge*

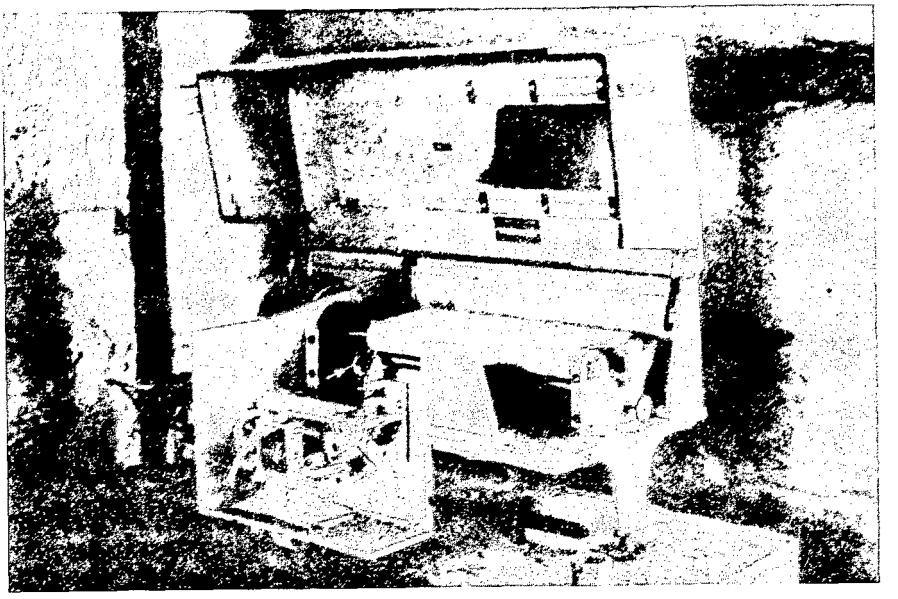


Figure 11. *"Winnipeg" type pressure gauge*

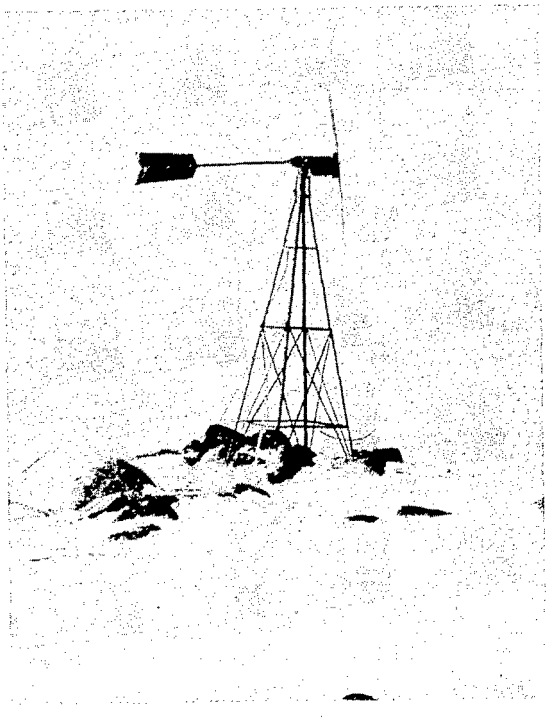


Figure 12. *Wind-driven generator set*

ıble 1. *Discharge measurements of Seal river below Great Island*

(1) Date	(2) Gauge height (metres)	(3) Area (m^2)	(4) Measured discharge (m^3/sec)	(5) Maximum velocity point[2] (m/sec)	(6) (5 x 3)	(7) Correction factor (4)/(6)	(8) Correction factor from curve	(9) Estimated final discharge from curve (m^3/sec)	(10) Percentage deviation (9) from (4)
963 :									
un 12	121.87	564.6	638	1.64	926	0.69	0.68	626	− 1.9
un 18	122.17	588.7	753	1.82	1070	0.70	0.69	734	− 2.6
ep 10	121.45	463.1	414	1.40	648	0.64	0.65	419	+ 1.2
964 :									
lar 20[1]	121.25	309.7	90.7	0.50	155	0.59	0.61	95.0	+ 4.5
un 18	121.44	510.6	476	1.41	720	0.66	0.66	472	− 0.8
965 :									
an 7[1]	121.72	401.8	150	0.65	261	0.57	0.62	161	+ 6.8
lar 24[1]	121.24	281.8	72.0	0.41	116	0.62	0.61	71.0	− 1.4
un 16	121.43	489.2	456	1.42	695	0.66	0.65	455	− 0.2
ul 26	121.31	470.6	389	1.31	616	0.63	0.64	396	+ 1.8
ep 20	120.88	388.7	247	1.04	404	0.61	0.62	249	+ 0.8
966 :									
lar 31[1]	121.04	300.4	86.5	0.46	133	0.63	0.62	85.0	− 1.8
un 22	122.75	723.5	1200	2.29	1600	0.72	0.71	1170	− 2.6
ul 13	122.44	651.9	960	2.13	1390	0.69	0.70	971	+ 1.1
967 :									
an 14[1]	121.04	333.9	134	0.63	210	0.64	0.61	129	− 3.9
lay 11[1]	120.88	279.9	86.8	0.48	134	0.65	0.61	82.5	− 5.0
un 25	122.65	665.9	1070	2.26	1510	0.71	0.70	1060	− 0.9
ul 18	122.59	692.9	1080	2.27	1570	0.69	0.71	1110	+ 2.7
					Mean:	*0.65*	*0.65*		

. Winter measurements.

. At vertical located 340 feet from initial point.

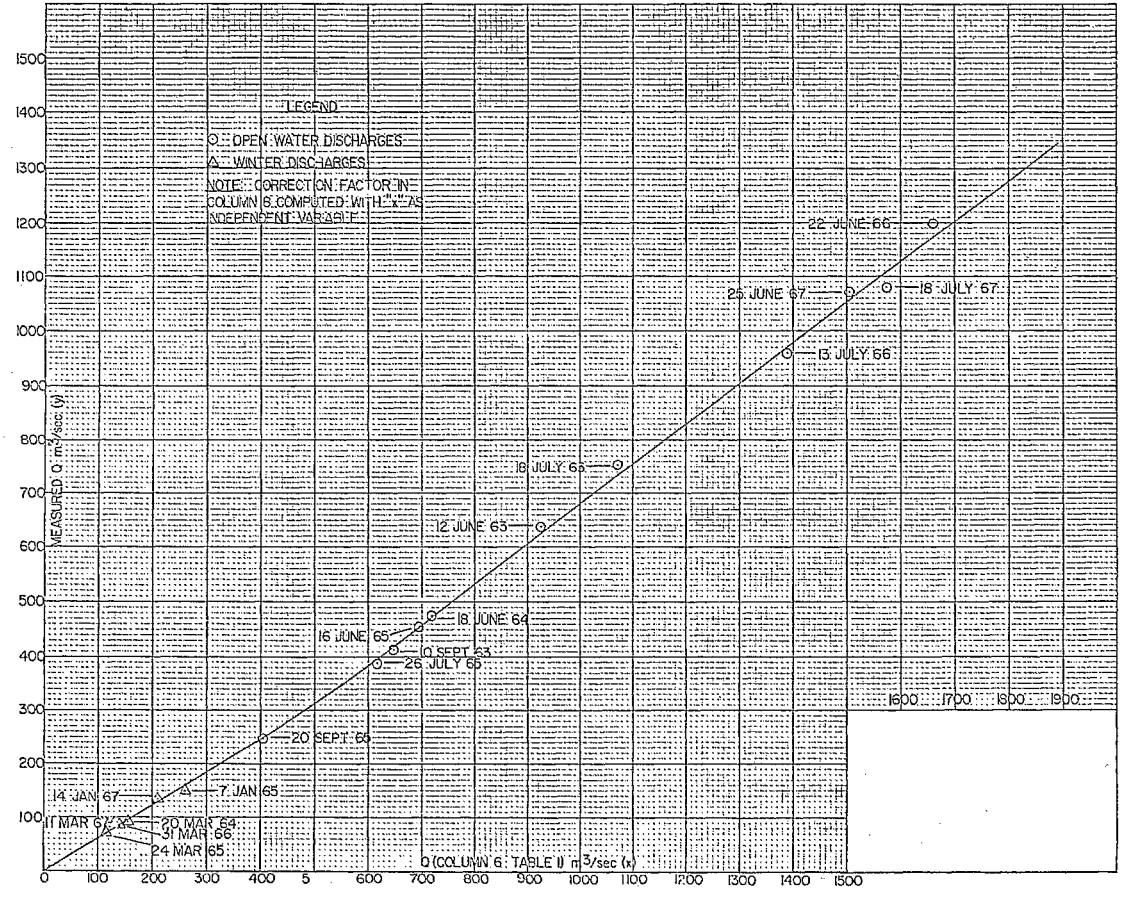


Figure 13. *Discharge curve relation for Seal river below Great Island*

The study made is only of a preliminary nature and no final conclusions can be drawn from it as yet.

Possible application of the above results, if proved correct

If differentiation in streamflow data requirements can be made between those south of latitude 55° from those north of this latitude, perhaps this alternative method for streamflow measurement could be adapted for the northern areas.

For example: It is possible that publication of "estimates" of flows to within ± 10 per cent accuracy is adequate for streams located north of latitude 55°.

To utilize the potential of this method fully, it is necessary to:

1. Develop an instrument which could be anchored firmly on the streambed to transmit velocities and possibly water levels
2. Develop an instrument which could be installed on the river bank to receive and record velocities and water levels transmitted by (1) above.

While the instrumentation described above is being developed, a possibility exists for the use of this method with conventional instruments for reduction of manpower requirements in hydrometric surveys in selected areas of Canada.

Estimation of streamflow under the ice cover

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Abstract. On watercourses where the ice cover is formed within a brief period of time, without previous ice run, the ratio of streamflow rates in the cross-section with and without ice cover (the factor φ) can be constructed on the basis of a sufficient number of streamflow measurements under ice. The curves thus obtained show appreciable differences, depending on the size and shape of the cross-sections, permitting no comparison and generalization. In the interest of generalization, a relationship has been established between the relative rates of streamflow in the ice-free cross-section and the factor φ.

The curves representing the relative streamflow against φ relationship are displaced relative to each other, but are of similar character. The family of curves determined in this manner from measurements can be used—under the climatic and hydrological conditions prevailing in Hungary—for constructing by interpolation the relationship between the φ value and stage, if a single streamflow value measured under ice is available and the hydrology of the cross-section under consideration is well known. The classification coefficient allowing for the geometry and hydrological conditions of the cross-section offers the possibility, under similar circumstances, for determining this relationship even without streamflow measurement under ice and for including the effect of changes in cross-section.

ESTIMATION DES DEBITS SOUS LA GLACE

Résumé. Sur les rivières sans débâcle où la couche de glace se forme très rapidement on peut déterminer le coefficient φ, rapport entre les débits de la section glacée et de la section libre de glace, en se basant sur une série de jaugeages de débit sous la glace. Les courbes déterminées diffèrent beaucoup selon la grandeur et les caractéristiques des sections ; elles ne sont pas comparables et on ne peut pas les généraliser. Les auteurs ont établi une relation entre les coefficients φ et les débits de la section libre de glace susceptible d'être généralisée.

Les courbes représentant la relation entre les débits relatifs et les coefficients φ sont parallèles. La famille des courbes déterminées de cette façon – dans les conditions climatiques et hydrologiques existant en Hongrie – peut être utilisée pour établir par interpolation la relation entre le coefficient φ et les hauteurs d'eau, si l'on possède une seule valeur de débit mesuré sous la glace et si la section est connue.

On peut déterminer cette relation et l'influence des modifications de la section à l'aide du coefficient classificateur A en considérant les propriétés géométriques et hydrologiques des sections sans qu'il soit nécessaire d'effectuer un jaugeage de débit sous la glace.

ESTIMACIÓN DE CAUDAL BAJO CAPAS DE HIELO

Resumen. En los cursos de agua, cuando se forma una capa de hielo por un breve periodo, sin que previamente fluyan témpanos, la relación de los valores de caudal en una sección transversal con la capa de hielo y sin ella, factor φ, se puede averiguar a partir de un número suficiente de mediciones de caudal bajo hielo. Las curvas así obtenidas muestran apreciables diferencias, en función del tamaño y forma de las secciones transversales, que no permiten comparación ni generalización alguna. En aras de la generalización, se ha establecido una relación entre los valores relativos del caudal en la sección libre de hielo y el factor φ.

Las curvas que representan la relación del caudal en función de φ, aparecen desplazadas, pero son de carácter similar. La familia de curvas determinadas de esta forma a partir de los datos de

medición se pueden usar –para las condiciones climáticas e hidrológicas que prevalecen en Hungria– para hallar por interpolación la relación entre el coeficiente φ y el nivel del agua, si se dispone de un valor único del caudal medido bajo hielo y las caracteristicas de la sección que se considera son bien conocidas. El coeficiente de la classificación, en función de la geometria y las condiciones hidrológicas de la seccion transversal, ofrece la posibilidad, bajo circunstancias similares, de determinar esta relación incluso sin medición del caudal bajo hielo y sin incluir el efecto de los cambios de seccion.

ОПРЕДЕЛЕНИЕ РАСХОДА ВОДЫ ПОД ЛЕДЯНЫМ ПОКРОВОМ

Резюме. На водотовах, где ледяной покров образуется за короткий период времени без предварительного ледохода, отношение расходов в поперечном сечении при наличии и отсутствии ледяного покрова, т. е. коэффициент φ может быть получен на основе достаточного числа измерений расхода подольдом. Кривые, полученные таким образом, показывают значительную разницу в зависимости от размера и формы поперечного сечения, не позволяя делать сравнение и обобщение. С целью возможности обобщения была установлена зависимость между относительным расходом в свободном ото льда поперечном сечении и коэффициентом φ.

Кривые, характеризующие зависимость относительного расхода в функции от φ, смещаются одна относительно другой. Общий характер семейства кривых, определяемых таким путем, может быть использован — при климатических и гидрологических условиях, превалирующих в Венгрии, — для построения путем интерполяции зависимости между значениями φ и уровнем, если имеется измеренное единичное значение расхода подольдом и гидрология данного поперечного сечения хорошо изучена. Возможно определение этой зависимости даже без измерения расхода подольдом, если будет сделана классификация коэффициента φ для соответствующих геометрических и гидрологических характеристик поперечного сечения.

An ice cover is formed almost every winter on the watercourses in Hungary and the stage-discharge relationship determined for ice-free periods is inappropriate for indicating the daily rates of streamflow in these periods. On the other hand, for the rational management of water resources, increasingly accurate information is required. The effects of the formation of the ice cover have therefore been investigated.

Ice cover forms on watercourses in Hungary in different ways, depending on the size of the stream. In large streams, such as the Danube and the lower reaches of the Tisza River, ice starts to form over upstream sections and the passage of floes is blocked where channel conditions are unfavourable. The ice cover grows gradually upstream, accompanied by an appreciable rise in water level and the development of a backwater.

On small watercourses, discharges diminish rapidly after the onset of the frost period, owing to reduced supply from the catchment, and the water is cooled at a high rate. The consequence is that minor watercourses tend to freeze over for their entire length within a very brief period of time, practically within a single day, without any noteworthy increase in stage. Subsequent considerations will be concerned with the influence of the ice cover on the rating curve of these small watercourses and with the possibility of allowing for the effect of the ice cover, relying on available data and without the need for flow measurements under the ice cover.

MEASUREMENTS PERFORMED

Using a standardized method of measurement and data processing, 272 streamflow measurements have been made in 146 profiles. For each of the streamflow values measured in the ice-covered profile, the stage was observed through a hole cut in the ice. The thickness of the ice submerged in water was used in the computations.

Once an ice cover was formed, its thickness increased rapidly, within a few days, to reach a final value of 0.35 m on average. Deviations from this value could, of course, be observed depending on the characteristics of the watercourse, variations in temperature and the duration of the frost period. The thickness of the ice cover developed was found to vary from 0.20 to 0.50 m.

For some of the gauging profiles (Tokaj, on the Tisza River) 20 to 30 year-long records of streamflow under the ice cover were available. In other profiles, measurements have been continued for 10 to 20 days a year (e. g. Gesztely on the Hernád River), while in the majority of cases one or two isolated measurements only were carried out under the ice cover.

Based on these measurements, a new "overfrozen" rating curve has been prepared for the ice period (Fig. 1) and from the results of daily measurements the changes were represented in the form of a hydrograph (Fig. 2). Using the rating curves applying to "overfrozen" and to ice-free conditions, the ratio $\varphi = Q'/Q$ has been computed for different rates of streamflow, where Q' and Q are streamflow rates in the overfrozen and ice-free profiles, respectively. In this manner rather simple realtionships could be obtained for some of the profiles (Fig. 1). The drawbacks of these curves are that they call for a great number of measurements, changes in the profile cannot be traced, the data obtained for a particular section cannot be compared with those of another section and, consequently, cannot be generalized for other profiles where only a small number of observations is available.

GENERALIZATION OF RESULTS

To obtain results of general validity, all measurements under ice were used to determine the factor φ, but rather than in terms of stage, these have been expressed in terms of the percentage fluctuation of streamflow and plotted thus against the value

$$Q\,[\%] = \frac{Q_i - Q_{\min}}{Q_{\max} - Q_{\min}} \cdot 100$$

(Fig. 3). In this manner smooth curves, showing no abrupt changes and unaffected by the shape of the profile, have been obtained for each gauging cross-section where sufficient data were available. These curves resemble each other and can thus be compared with curves constructed in a similar manner for any other watercourse.

In Fig. 3 the value of the factor φ determined from individual measurements has been indicated by circles along with the name of the particular gauging profile; this family of curves facilitates the construction of the appropriate φ curve from a single streamflow measurement under ice. This, in turn, can be used for estimating streamflow in the ice-covered section in terms of stage, provided that the rating curve for the particular section is available and the extreme streamflow rates are reasonably well known. In practical applications, the above method is used for constructing the "overfrozen" rating curve, from which the streamflow under the ice is obtained in terms of the daily stage.

THE INFLUENCE OF FACTORS PARTICULAR TO THE PROFILE

The curves illustrated in Fig. 3 show a certain regularity, which depends presumably on the geometry of the cross-section. Attempts have been made to determine this regularity to check results derived from a small number of measurements under ice, to estimate the reduced streamflow values required for streamflow statistics and also for gauging profiles where no such measurements are available.

All dimensional characteristics of the cross-section have been taken into consideration; these can be obtained readily from the data of the ice-free section and any change would entail a change in the factor φ. The influences of changes in water temperature and changes in roughness conditions are combined in the results, and are presumably proportional to the mean velocity v_K in open flow conditions.

The effects of changes in the wetted cross-sectional area and in roughness conditions have been expressed in terms of the mean depth, m_K, surface width, S, the cross-sectional area, F, and thickness of the ice cover, d. Under conditions prevailing in Hungary, the thickness of the ice cover once formed may be taken as approximately identical for any profile, neglecting the periods of ice-cover formation and break-up of a few days' duration, and considering only the thickness of ice covers existing for extended periods.

The magnitude of φ is directly related to m_K and $\sqrt{F}$, while it is inversely proportionate to S, v_K and d. An expression of the form

$$A = \frac{m_k \sqrt{F}}{S \cdot v_k \cdot d}$$

was found most suitable for describing the geometrical properties of the ice-free cross-section.

The factors controlling the magnitude of the coefficient A change with changes in stage. Difficulties are introduced in practical applications by this conflicting variation of the coefficient A, and thus it is believed advisable to adjust the magnitude of A—on the basis of existing data—to the most characteristic relative rate of streamflow. Hence the magnitude of A has always been computed with data from sections appertaining to a relative streamflow of 4 per cent. This value has been adopted because under the conditions prevailing in Hungary it is characteristic of winter run-off, and the characteristic point of the φ curves is also around this relative streamflow percentage.

The dimensions of A are the reciprocal of the dimensions of velocity ; the magnitude of A is proportional to the reciprocal of a velocity representative of the cross-section. However, owing to the absence of adequate data, no physical interpretation has been attributed to the coefficient A, which has been used only as a coefficient characteristic for the cross-section.

Values of A have been determined for the relative streamflow $Q = 4$ per cent in 23 gauging profiles for which detailed hydrological information was available. These are shown in Fig. 4. The position of the points in the figure demonstrates the correctness of the classification coefficients.

The 23 sets of data have been correlated and the relationships $\varphi = m \,.\, A^{1/3}$ and $m = 0.462$ have have been obtained. The scatter of the measurements about the curve is $\Delta\varphi = 0.038$, corresponding to a standard deviation of 7.3 per cent related to the average $\varphi_a = 0.52$.

Hereafter, using the diagrams in Fig. 3, the φ versus A relationships have been constructed for different Q percentages (Fig. 5). The magnitude of A has been

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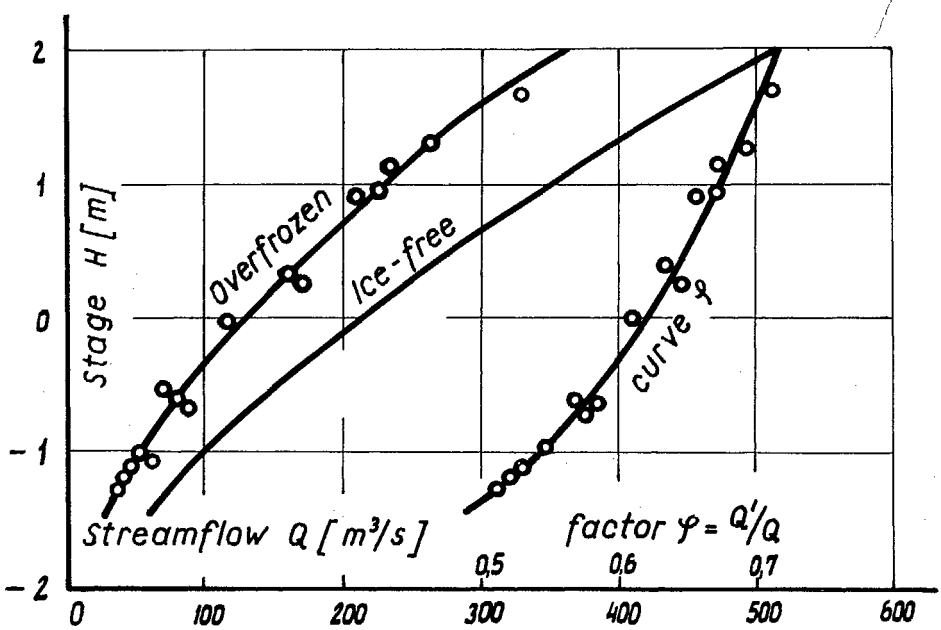


Figure 1. *Ice-free and "overfrozen" rating curves for the gauging section Tokaj on the Tisza river*

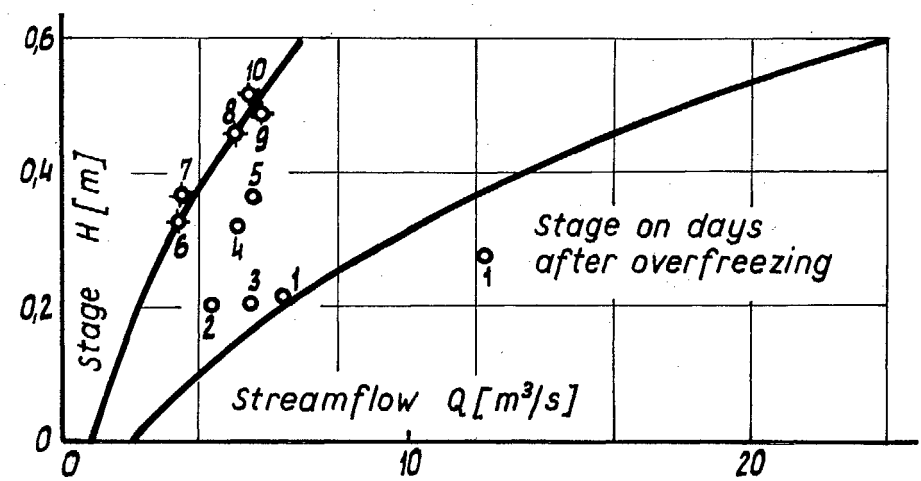


Figure 2. *Changes in streamflow rates under the ice cover in the Gesztely gauging section on the Hernád river*

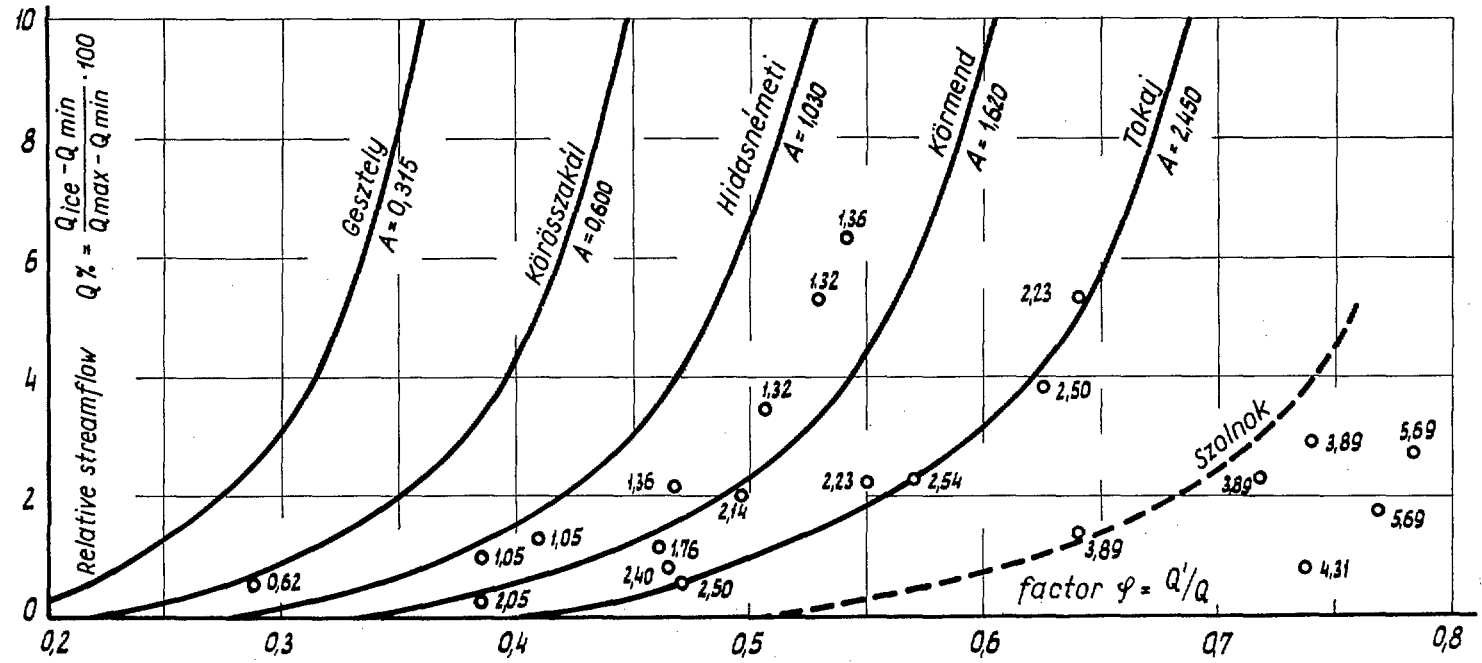


Figure 3. *Variations of φ versus relative streamflow (Q %)*

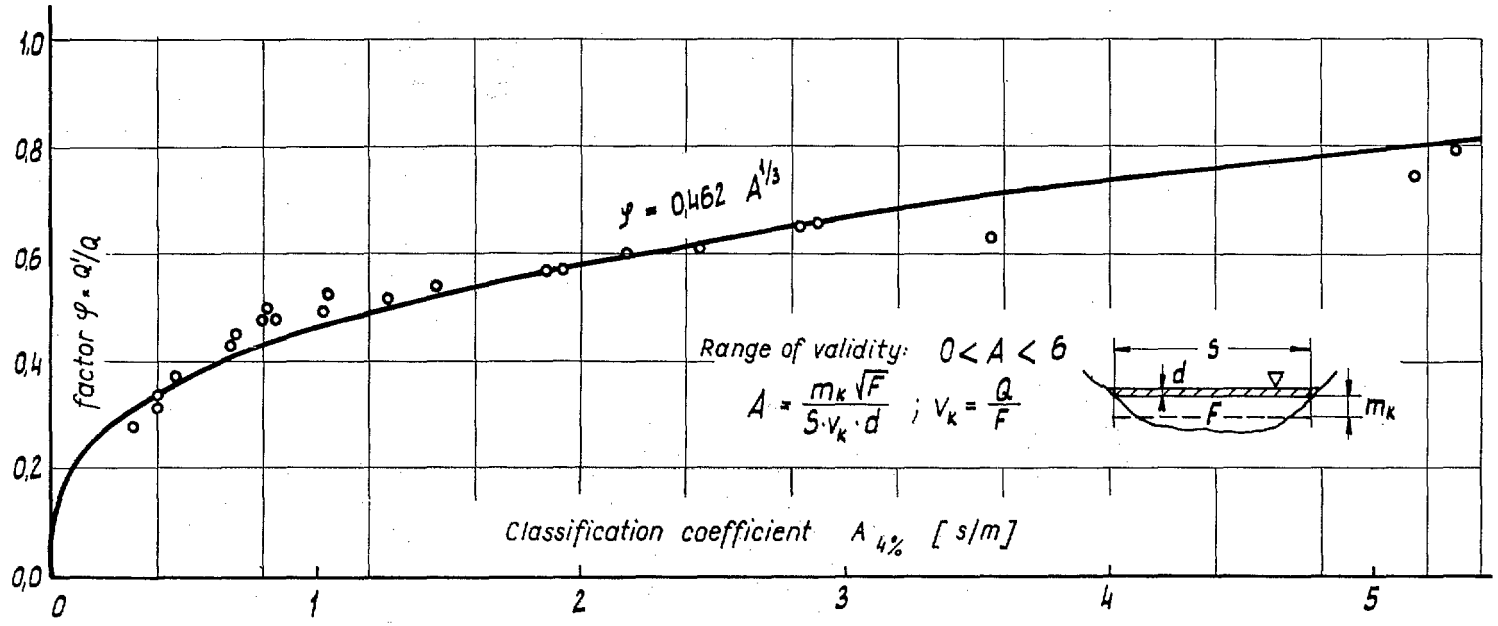


Figure 4. *Variations of φ versus the classification coefficient A at 4 per cent relative streamflow (Q_4 %)*

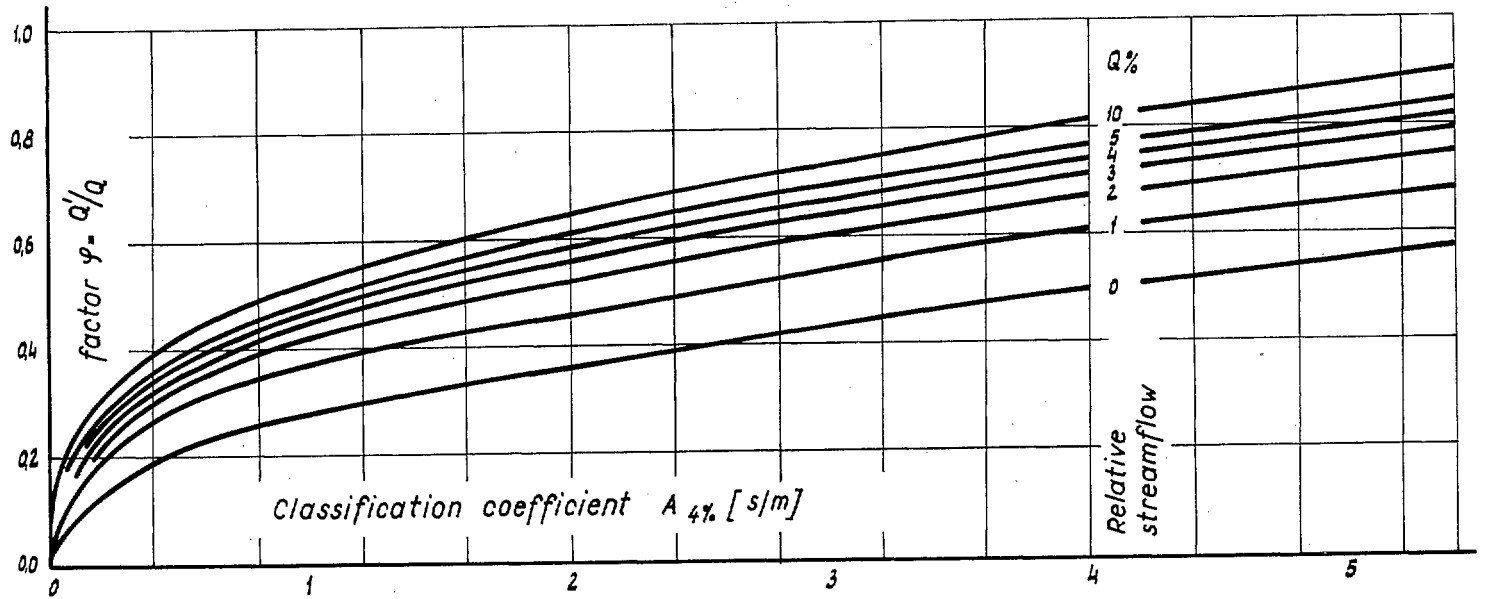


Figure 5. *Variations of φ versus the classification coefficient appertaining to a relative streamflow of 4 per cent, at different relative streamflow rates*

computed, however, always from the data of the ice-free cross-section appertaining to the relative streamflow of 4 per cent, and has been considered constant within a cross-section, regardless of the actual streamflow. On the other hand, in the case of variable beds the magnitude of A has always been computed using the new cross-section data.

If the coefficient A is known, each vertical through the family of curves will yield the values of the φ curve for the particular cross-section and the characteristic rating curve can be constructed without actual streamflow measurements under ice, provided that the hydrology of the watercourse is sufficiently known for the particular cross-section.

CONCLUSIONS

On minor watercourses, the ratio of streamflows conveyed under an ice cover to those at open surface conditions, namely the factor φ, can be determined in a most reliable manner by streamflow measurements under ice, performed regularly. However, the relationship thus determined between the factor φ and stage applies only to the period of measurement and cannot be generalized. When the values are represented in terms of the apparent (ice-free) streamflow, the curves obtained for watercourses of different magnitude and character will be of similar shape but differ in position from each other. This family of curves can be used for constructing by interpolation the φ against $Q\,\%$ relationship on the basis of a single streamflow measurement under ice in the particular cross-section.

The classification coefficient allows for the geometry of the different cross-sections and lends itself to the construction of a family of curves, yielding information on the variations of φ even if no measurement under ice is performed. For this purpose the maximum and minimum rates of streamflow must be known with a fair degree of reliability and the variation of streamflow and wetted cross-sectional area with stage must be available.

Checks on the φ factors were run in cross-sections situated above the confluence of the Tisza and Bodrog rivers and in one downstream of the confluence on the Tisza River. For these sections the total run-off volume was computed for the full ice-cover period on the basis of apparent streamflows and the computed φ values. The run-off downstream of the confluence estimated from apparent streamflow rates was smaller than the total obtained for the two upstream sections. Estimating the same values on the basis of the φ factor, the results were found to be in fair agreement.

II.2 Tracer methods
Jaugeage par traceurs

Les jaugeages par la méthode de dilution en 1970

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Résumé. La méthode de mesure des débits de liquide par dilution est actuellement très utilisée dans de nombreux pays.

Son domaine d'application est très étendu : il n'est plus seulement celui des faibles débits, puisqu'un jaugeage de 2 000 m^3/s a été réussi en 1968, ni celui des eaux propres puisque de nombreuses mesures ont été effectuées dans des eaux chargées de matières en suspension ou contenant des impuretés de toutes sortes.

Deux procédés sont actuellement utilisés : le procédé par injection à débit constant et le procédé par intégration.

Le traceur peut être choisi parmi une liste étendue dont le plus utilisé en France est le bichromate de soude, mais on peut citer la rhodamine, le chlorure de sodium, le chlorure de lithium.

Dans le domaine des radio-isotopes, les éléments les plus utilisés sont le brome-82 et l'iode-131, mais l'apparition de "générateurs de radio-élément" offre de nouvelles possibilités très intéressantes.

Le contrôle permanent du débit par emploi d'un générateur à étain indium commence à pouvoir être envisagé et fait l'objet de premières expériences.

GAUGING BY THE DILUTION METHOD IN 1970

Abstract. The dilution method is frequently used nowadays in many countries for the measurement of liquid discharge rates.

This method has a very wide field of application: its use is no longer restricted to low discharges, since a 2,000 m^3/s flow was successfully gauged in 1968; nor is it restricted to clean water, since many measurements have been made in water containing material in suspension or various kinds of impurities.

Two techniques are employed at present: the constant discharge process and the integration process.

The tracer can be selected from a wide range: sodium bichromate is the most commonly used in France, but mention must also be made of rhodamine, sodium chloride and lithium chloride.

In the field of radioisotopes, the most commonly used elements are bromine 82 and iodine 131, but the appearance of "radio-element generators" opens up some extremely interesting new possibilities.

It is now possible to begin to envisage continuous discharge measurements using an indium-tin generator, and preliminary experiments have been carried out.

AFORO POR EL MÉTODO DE DILUCIÓN EN 1970

Resumen. El método de dilución se utiliza actualmente con frecuencia en muchos países para la medición de los valores de caudal líquido.

Este método tiene un campo de aplicación muy amplio: su utilización no queda limitada a los caudales bajos, puesto que en 1968 se aforó con éxito un caudal de 2 000 $m^3/slg.$; ni tampoco queda restringido a aguas limpias, puesto que se han realizado muchas medidas en aguas con materiales en suspensión o diversos tipos de impurezas.

Actualmente se emplean dos técnicas: el método de caudal constante y el método de integración.

El trazador se puede seleccionar dentro de una amplia gama: el dicromato sódico es el utilizado más comúnmente en Francia, pero se debe hacer mención también de la rhodamina, el cloruro sódico y el cloruro lítico.

En el campo de los radioisótopos, los utilizados más comúnmente son el bromo 82 y el yodo 131, pero la aparición de "generadores de radioelementos" abre algunas nuevas posibilidades extremadamente interesantes.

Ahora es posible comenzar a vislumbrar mediciones de caudal continuo utilizando un generador de indio estaño, con el que se han realizado ya los primeros experimentos.

ИЗМЕРЕНИЯ РАСХОДОВ МЕТОДОМ СМЕШЕНИЯ В 1969 Г.

Резюме. Метод смешения в настоящее время используется часто во многих странах для измерения расходов жидкости.

Он имеет весьма широкую область применения : его применение не ограничивается теперь измерением низких расходов, поскольку расход в 2000 Ф³/сек. (~ 57 м³/сек.) был успешно измерен в 1968 г.; не ограничивается также измерением в условиях чистой воды, т. к. многие измерения были выполнены в воде, содержащей материал во взвеси или различные виды загрязнений.

В настоящее время разработаны два метода : способ с постоянным расходом и интеграционный способ.

Индикатор может быть выбран в широком диапазоне : бихромат натрия — обычно используется во Франции, но следует упомянуть также родамин, поваренную соль и хлористый литий.

В области радиоизотопов наиболее часто используются такие элементы как бром 82 и иод 131, но появление « генераторов радиоэлементов » открывает некоторые чрезвычайно интересные новые возможности.

Появилась возможность исследовать вопрос о непрерывных измерениях расхода с использованием генератора индий-олово; первые эксперименты выполняются.

I. AVANT-PROPOS

Les méthodes de mesure des débits des liquides par dilution, créées en France en 1863, n'ont pris un réel essor qu'en 1950, au Laboratoire d'hydraulique de l'Institut polytechnique à Grenoble, sous l'impulsion de H. Dumas, M. Dodero et de la Région d'équipement hydraulique Alpes II, à Chambéry. Pendant longtemps, on a considéré que ces méthodes n'étaient applicables qu'à la mesure des petits débits de torrents turbulents.

La plupart des pays utilisent actuellement de façon courante ces mesures par dilution, même dans les cas nécessitant une grande précision, avec des traceurs divers et pour des gammes de débits variées.

En 1970, grâce aux nouvelles méthodes utilisées, à l'expérience acquise et aux progrès réalisés, on peut envisager l'application de ces procédés par dilution à de très nombreuses rivières et, dans certains cas favorables, pour des débits de plusieurs milliers de m³/s (2 200 m³/s ont été jaugés ainsi à Madagascar sur la Betsibooka.

Ces methodes permettent aussi les mesures des débits dans des rivières relativement tranquilles, à des vitesses lentes de 0,10 à 0,50 m/s. Des expériences concluantes à ce sujet ont été menées, plus particulièrement en Hongrie et en France.

La turbidité des eaux constituait un obstacle majeur pour la réalisation de tels jaugeages, en particulier lorsque l'analyse était conduite par des méthodes colorimétriques. Même dans ce cas il est possible maintenant d'obtenir des résultats satisfaisants par filtration et, en outre, l'utilisation de traceurs radio-actifs permet de s'affranchir des matières en suspension.

Les cahiers des charges des machines hydro-électriques font maintenant mention, dans plusieurs pays, de l'utilisation officielle des procédés par dilution pour la mesure des débits dans le contrôle des rendements.

II. PROCEDES UTILISES

Deux procédés par dilution sont actuellement utilisés : injection à débit constant et intégration.

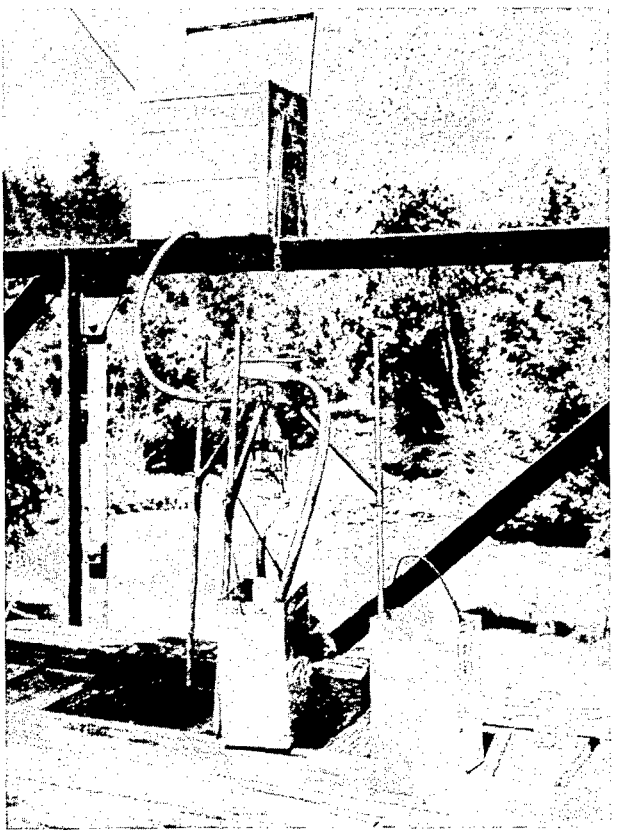


Figure 1. *Matériel d'injection à débit constant : cuve et vase à niveau constant petit modèle*

II.1 Procédé par injection à débit constant (fig. 1)

On injecte, en un point judicieusement choisi pour favoriser le mélange rapide avec le débit Q à mesurer, un débit constant q d'une solution de traceur à la concentration C_1 pendant un temps suffisamment long pour obtenir un palier de concentration C_2 dans la rivière, en une section assez éloignée pour que le mélange de ce produit avec la rivière soit réalisé, ce qui se traduit dans ce cas par la constance de la concentration C_2 dans toute la section de prélèvement pendant le palier de concentration.

La formule exprimant que la masse de traceur injecté par seconde est égale à la masse de traceur passant par seconde dans la rivière durant le palier de concentration : $QC_2 = qC_1$ conduit à la valeur du débit $Q = q\ C_1/C_2$.

L'analyse au laboratoire consiste à déterminer la valeur du rapport $\frac{C_1}{C_2}$ sans qu'il soit nécessaire de connaître ni C_1 ni C_2, en comparant les échantillons prélevés dans la rivière à la concentration C_2 à une gamme de dilution réalisée en laboratoire avec de l'eau de la rivière, à partir d'un échantillon de la solution injectée C_1.

II.2 Procédé par intégration

On injecte de façon quelconque, dans la rivière de débit Q, un volume connu V d'une solution de traceur à la concentration C_1.

On prélève des échantillons pendant tout le temps de passage T du traceur dans une section située suffisamment à l'aval pour que le bon mélange soit réalisé, ce qui se traduit dans ce deuxième procédé par la constance de l'intégrale :

$$\int_o^T c_2 \, dt$$

quel que soit le point de prélèvement dans la section.

On démontre que les expressions :

$C_2 = C^{te}$ dans le premier procédé par injection à débit constant,

et $\int_o^T c_2 \, dt = C^{te}$ dans le deuxième procédé par intégration, sont équivalentes et que les distances de bon mélange sont théoriquement les mêmes dans les deux cas.

Le débit est obtenu par la relation suivante qui exprime la conservation de la quantité de traceur :

$$Q = \frac{VC_1}{\int_o^T c_2 dt} = \frac{VC_1}{C_2 T}$$

c_2 est la concentration d'un échantillon prélevé à l'instant t et C_2 la valeur moyenne pendant le temps T de tous les échantillons prélevés ou d'un échantillon moyen prélevé à débit constant en un point de la section aval.

L'analyse est faite de façon analogue à l'analyse d'un jaugeage avec injection à débit constant en déterminant, par comparaison, des échantillons prélevés avec une gamme de dilution :

$$\frac{C_1}{C_2} \quad \text{ou} \quad \frac{C_1}{\int_o^T c_2 \, dt}$$

II.3 Distance de bon mélange. Temps de passage des nuages

Les néophytes, en matière de jaugeage par dilution, sont le plus souvent inquiets des difficultés pratiques de détermination des distances de bon mélange et des durées de passage des injections instantanées. On trouve alors ce souci majeur d'établir des formules permettant le calcul de ces valeurs d'après les caractéristiques géométriques ou hydrauliques de la rivière.

Certaines formules sont très simples, d'application aisée et ne fournissent que des ordres de grandeur. Les formules les plus élaborées conduisent souvent à des résultats plus précis, mais introduisent des paramètres variés et complexes dont la détermination exige presque une mesure de débit préalable.

Enfin, les caractéristiques des rivières varient de façon trop importante tout au long des tronçons de mesures pour permettre l'approche de ce problème de façon mathématique.

En pratique, lorsque les débits sont faibles (quelques dizaines de m^3/s) et les eaux claires, la meilleure méthode de détermination des distances de bon mélange et du temps de passage consiste à jeter un colorant de façon instantanée au lieu prévu pour l'injection, à surveiller sa répartition et à noter son temps d'étalement à la section choisie pour les prélèvements.

Dans le cas où ces essais sont impossibles, on peut faire des essais avec des flotteurs partiellement immergés ou tenir compte d'ordres de grandeur apportés par l'expérience en fonction de la largeur de la rivière, et figurant dans le tableau ci-dessous :

	Largeur de la rivière			
	Quelques mètres	Quelques dizaines de mètres	50 à 200 mètres	Plus de 200 mètres
Distance de bon mélange	50 à 300 m	300 à 2000 m	1 000 à 15 000 m	Cas particulier
Temps de passage d'injection instantanée	5 à 20 mn	10 à 30 mn	15 à 60 mn	La méthode est souvent de mise en œuvre difficile

Il faut retenir que le bon mélange est plus difficile à obtenir transversalement que dans le sens vertical. Les facteurs de bon mélange sont les courants transversaux, les rétrécissements (une distance de 3 à 4 km a été suffisante sur la Betsibooka, large de 400 m, parce que le tronçon de mesure présentait un court rétrécissement de 50 m de largeur).

II.4 Avantages et inconvénients des deux procédés

Le procédé par injection à débit constant est le plus ancien ; il a donc fait ses preuves et le matériel nécessaire est parfaitement au point : il est plus sûr et souvent plus précis, car on analyse une dizaine d'échantillons indépendants théoriquement identiques : la moyenne des valeurs C_1/C_2 trouvée pour ces dix échantillons, dont les

analyses sont entachées d'erreurs accidentelles, a une probabilité plus forte d'être plus proche de la vérité que la valeur C_1/C_2 de l'échantillon moyen du procédé par intégration.

Il est plus facile de vérifier la condition de bon mélange dans la section de prélèvement dans le cas du procédé par injection à débit constant ($C_2 = C^{te}$) que dans le procédé par intégration $\left(\int_0^T c_2 \, dt = C^{te}\right)$.

Il n'est pas très facile de déterminer avec précision le temps T. Dans le cas du procédé par injection à débit constant, il suffit d'injecter pendant un temps suffisamment long et de prélever un grand nombre d'échantillons pour obtenir un résultat utilisable ; dans le cas du procédé par intégration, un prélèvement moyen trop court conduit à un débit erroné, un prélèvement trop long conduit à des concentrations trop faibles pour l'analyse.

Lorsque les berges sont susceptibles de capter une partie du traceur (cas des berges enneigées ou verglacées en particulier), le procédé par injection à débit constant semble préférable, car les berges se saturent alors rapidement à la concentration C_2 et le palier est vite obtenu sans erreur, alors que la désaturation est un phénomène très lent conduisant, dans le cas du procédé par intégration, à prélever pendant un temps beaucoup trop long, entraînant des concentrations moyennes très faibles.

Au contraire, le procédé par intégration présente les avantages suivants :

a) Matériel plus simple, réduit le cas échéant à un récipient pour la solution injectée, une boîte de prélevements et une montre (fig. 2).

b) Gain de temps : on peut injecter la solution de façon presque instantanée ; la durée de la mesure est réduite à la somme du temps de parcours du traceur, entre les sections d'injection et de prélèvement, et de son temps de passage au lieu de



Figure 2. *Matériel de prélèvement : bonbonne d'eau de la rivière et caisse de prélèvement (échantillons)*

prélèvement, alors que, dans le cas d'une injection à débit constant, il faut ajouter à ces valeurs le temps d'installation du matériel (de 15 à 30 mn) et la durée choisie pour le palier de concentration (en général 15 mn).

c) Gain de traceur : si T est le temps de passage à la section de prélèvement du traceur injecté de façon instantanée et si θ est la durée choisie pour la durée du palier de concentration dans le cas du procédé par injection à débit constant, les poids de traceur nécessaires sont dans le rapport :

$$\frac{T}{T+\theta}$$

Le procédé par intégration est donc d'autant plus économique que le temps de passage T est court. Ainsi, pour certains torrents, le temps T est inférieur ou égal à 5 mn, θ est souvent choisi égal à 15 mn :

$$\frac{T}{T+\theta} = \frac{5}{20} = \frac{1}{4}$$

Il faut donc quatre fois moins de traceur dans le procédé par intégration que dans le cas de l'injection à débit constant.

Sur des rivières lentes et larges, au contraire, les temps T de passage du traceur sont compris entre 30 mn et une heure, $T(T+\theta)$ varie de $\frac{30}{30+15}$ à $\frac{60}{60+15}$; les gains de traceur ne sont que de 20 à 35 %.

d) Gain de temps pour l'analyse d'une série de jaugeages ; lorsqu'une série de jaugeages est faite au cours d'un même déplacement sur des rivières aux eaux limpides et de caractéristiques chimiques voisines, il est intéressant, dans le cas du procédé par intégration, d'injecter dans les diverses rivières des volumes différents d'une même solution à la concentration C_1 maximale : on peut ainsi, à l'analyse, comparer les échantillons prélevés dans toutes les rivières à une gamme unique de

En pratique, cela n'est pas toujours possible dans le procédé par injection à débit constant, car il n'est pas facile d'injecter un débit constant bien connu inférieur à 10 cm^3/s et il est nécessaire de faire varier la concentration C_1 injectée d'une rivière à l'autre pour obtenir des concentrations C_2 convenables pour l'analyse ; d'une part, les poids de sel étant plus grands dans ce cas, on préfère emporter le traceur solide et non en solution.

III. COMPARAISON DES MESURES AU MOULINET ET DES JAUGEAGES PAR DILUTION

III.1 Précision

De nombreux essais comparatifs des deux méthodes faits par la DTG lors des dix dernières années montrent que les écarts dépassent très rarement 2 à 3 % quand les méthodes sont appliquées avec soin, dans de bonnes conditions, mais que les débits obtenus par des mesures au moulinet sont en général plus faibles que ceux obtenus dans les jaugeages par dilution.

Trois raisons au moins permettent d'expliquer le sens de cet écart : *a*) le moulinet ne s'améliore pas, mais se détériore dans le temps, donc tendrait à sous-

estimer les vitesses ; *b*) la méthode de dilution prend en compte une partie de l'écoulement à travers les graviers qui n'est pas mesurée au moulinet ; *c*) il ne peut pas y avoir en général apparition, mais uniquement disparition de traceur entre les sections d'injection et de prélèvement (absorption par les berges ou les matières en suspension, réduction chimique . . .) ; la méthode de dilution conduit alors à surestimer le débit.

III.2 Mise en œuvre

Les méthodes sont le plus souvent complémentaires, car si les mesures au moulinet conviennent mieux dans les écoulements à filets parallèles, la méthode de dilution elle, convient plus particulièrement aux régimes turbulents.

En règle générale, on réservera les mesures au moulinet pour les rivières les plus larges et les plus régulières, lorsque la distance nécessaire au bon mélange, dans le cas des méthodes de dilution, devient exagérée (plus de 3 km par exemple).

Avantages des mesures au moulinet. On mesure le débit dans une section et non dans un tronçon de rivière ; on ne prend donc pas de risques dans les zones de fuites et l'on peut jauger une rivière entre deux affluents même rapprochés. On n'introduit pas de produits dans la rivière.

Le dépouillement d'un jaugeage est très rapide lorsqu'il a été fait sur le terrain par intégration continue (une demi-heure), où lorsqu'on utilise un calculateur électronique.

Avantages des méthodes de dilution. On n'introduit pas de matériel fragile dans l'écoulement. La partie efficace du jaugeage est très rapide (5 à 20 mn) ; le débit varie donc peu pendant les mesures en période de crues et l'on a constaté que les résultats étaient cohérents en régime transitoire, lorsque les variations du débit étaient faibles pendant la mesure.

Avec une seule injection et plusieurs lieux de prélèvements, on peut mesurer les débits d'une rivière et de ses affluents.

Les mesures sont possibles dans toutes les conditions de turbulence, même pour de faibles ou de très fortes vitesses. Les rivières tranquilles se prêtent bien aux jaugeages par dilution, contrairement à ce qu'on pensait initialement, dans la mesure où elles ne présentent pas de zones d'eau morte et où elles ne sont pas trop larges.

IV. TRACEURS UTILISES

IV.1 Traceurs non radio-actifs

Le bichromate. C'est le traceur le plus utilisé en France en 1970. Non toxique aux concentrations utilisées, très soluble dans l'eau (600 g par litre), assez peu onéreux (2 F le kg, hors taxes), peu fréquent dans les eaux naturelles, analyse colorimétrique à de faibles concentrations facile pour des agents non chimistes. De plus, il est assez stable, bien que le Cr + 6 puisse être réduit en Cr + 3, ce qui est un inconvénient dans les eaux industrielles.

L'analyse colorimétrique, qui exige l'introduction préalable d'un réactif, est faite à des concentrations comprises entre 0,2 et 2 mg de bichromate par litre, ce qui conduit à utiliser normalement des doses de 0,2 à 2 kg de bichromate par m^3/s à jauger (l'analyse colorimétrique utilisée est une méthode de zéro consistant à com-

parer les échantillons prélevés à des dilutions témoins que l'on sait préparer avec précision, traitées de la même façon)

Lorsque les débits sont importants et conduiraient à injecter des poids de sel exagérés, on peut utiliser des doses dix et même cent fois plus faibles et opérer une reconcentration des échantillons et des dilutions témoins avant l'analyse colorimétrique. Un procédé simple utilisable par des techniciens polyvalents a été mis au point par la DTG ; il consiste à recueillir le complexe coloré bichromate + réactif dans un volume n fois plus faible de butanol par séparation de phase. On fait subir le même traitement aux prélèvements effectués dans la rivière et aux dilutions témoins. Le phénomène étant fidèlement reproductible, on peut alors utiliser la méthode colorimétrique classique. C'est ainsi que pour un jaugeage de 1 000 m^3/s, il serait possible actuellement de n'utiliser que 10 à 20 kg de bichromate, donc 20 à 40 litres de solution.

On peut l'utiliser même en eaux chargées en matières en suspension, en filtrant sous vide échantillons et dilutions témoins avec des filtres plastiques à pores de 1 à 2 μ, en prenant soin de saturer préalablement chaque filtre avec 10 cm^3 de l'échantillon à filtrer.

Les rhodamines. Plusieurs types de rhodamines sont principalement utilisés en 1970 : B, Wt et sulfo-rhodamine G. Dans tous les cas, il s'agit d'un colorant puissant analysable à de très faibles concentrations (10^{-9} et même 10^{-10}) par fluorescence, peu soluble dans l'eau (10 à 50 g par litre), assez stable, mais absorbé par les argiles (prix de 30 à 100 F le kg suivant les qualités). Peu fréquent dans les eaux naturelles, pas toxique aux concentrations utilisées, mais très salissant.

L'avantage principal de ce traceur est la possibilité d'analyse directe, sans réactif, par fluorescence, à de très faibles concentrations, ce qui permet, avec un fluorimètre enregistreur portatif (type Turner par exemple), de déceler son passage et de faire la mesure *in situ.*

Inconvénients principaux : sa faible solubilité, ce qui exige des volumes de solution importants, son absorption par les argiles, son pouvoir salissant.

Le chlorure de lithium. Nouveau venu dans la gamme des traceurs, son expérimentation industrielle en France et au Royaume-Uni n'a commencé, à notre connaissance, que récemment.

Corps assez stable, très léger, très soluble dans l'eau (600 g au litre),ce traceur est analysable assez facilement à 1 mg de ClLi par litre, par spectrométrie de flamme en émission ; prix voisin de 30 F par kg, peu fréquent dans les eaux naturelles, non toxique aux concentrations utilisées.

L'avantage principal de ce traceur est sa stabilité, même en eaux industrielles ; son inconvénient majeur est son prix dans les conditions usuelles d'utilisation en 1970, mais on peut espérer utiliser ultérieurement des doses plus faibles.

On peut citer, pour mémoire, d'autres traceurs rarement utilisés : le chlorure de sodium (résistivité et analyse chimique), le sulfate de manganèse et le nitrite de soude ; enfin, la fluorescéine, qui connaît une certaine faveur au Japon pour les mesures en conduite, mais dont l'utilisation en rivière semble peu recommandable en raison de sa grande instabilité à la lumière solaire.

IV.2 Analyse par activation

Certains traceurs, tels que l'iodure de sodium, peuvent être analysés par activation.

Ces procédés font des progrès continuels et, bien qu'ils ne paraissent pas encore concurrentiels avec les analyses colorimétriques ou par spectrométrie de flamme, on peut espérer que ces méthodes d'analyse complèteront bientôt utilement les procédés actuellement en vigueur, en permettant l'injection dans les rivières de traceurs non radio-actifs, tout en bénéficiant des possibilités de comptage offertes par les radio-isotopes.

IV.3 Radio-isotopes

Traceurs à rayonnement γ. Les traceurs à rayonnement γ les plus employés sont le brome-82 (36 heures), l'iode-131 (8 jours), l'or-198 (2,7 jours) et le sodium (15 heures).

Parmi les avantages de ces radio-isotopes, par comparaison avec des traceurs minéraux ou organiques, on peut citer : *a*) la facilité et la précision du comptage, à partir de sondes à scintillation ou de compteurs de Geiger-Muller ; *b*) la détection *in situ* par ces mêmes appareils ; *c*) le fait que le prix du traceur à utiliser n'est pas proportionnel au débit à mesurer, ce qui est intéressant pour les gros débits ; *d*) l'absorption par les matières en suspension et les réductions chimiques ne perturbent pas le comptage.

En revanche, on peut citer comme inconvénient : *a*) la nécessité de formalités administratives, ce qui conduit à ne pas utiliser ce type de traceur de façon imprévue, en particulier lors des jaugeages de crues ; *b*) l'évolution de l'activité du traceur dans le temps : du point de vue de la sécurité, on a intérêt à utiliser des traceurs à période courte, mais on dispose alors de peu de temps entre la livraison du traceur et la

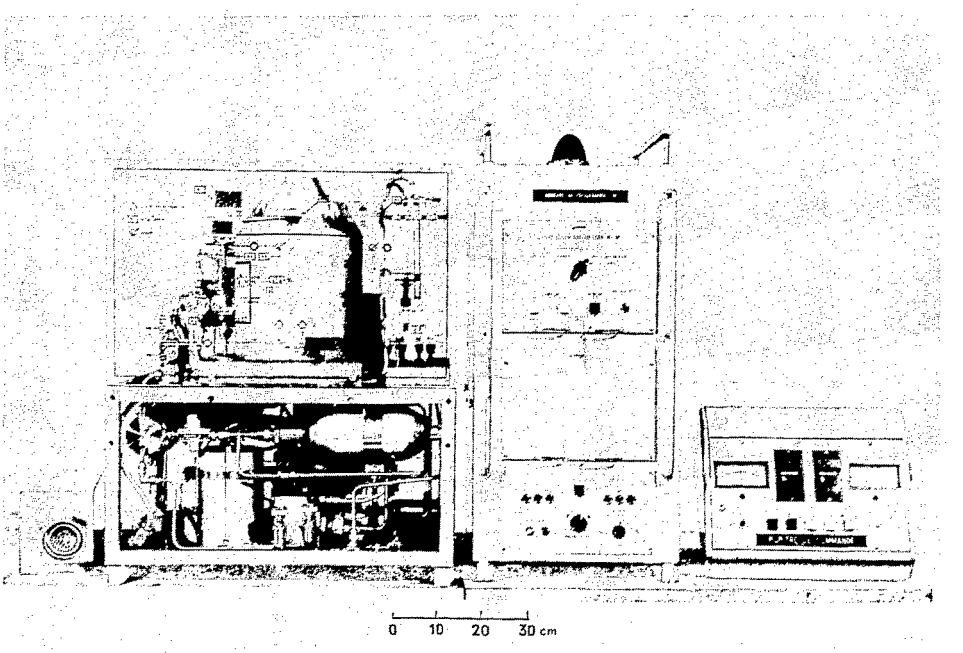


Figure 3. *Générateur de radio-élément pouvant utiliser indifféremment un couple étain-indium ou césium-baryum*

mesure qui doit donc avoir lieu à faible distance du centre d'irradiation ; *c*) le poids des enceintes de protection en plomb ; *d*) la nécessité de personnel spécialisé et habilité.

Traceurs à rayonnement β. L'utilisation de traceurs à rayonnement β comme le tritium est séduisante, car ces radio-isotopes sont transportables dans des ampoules scellées sans protection particulière, et un faible volume peut être utilisé pour mesurer des débits importants. En outre, le tritium, par sa nature même, ne présente pas de risque d'absorption par les matières en suspension.

Quelques inconvénients importants limitent son emploi : la durée et la difficulté de l'analyse, donc son prix ; l'impossibilité de le détecter et de le mesurer *in situ* ; sa trop longue période de douze ans liée au risque de contamination des puits ; le fait qu'il est utilisé par les hydrogéologues pour le tarage des nappes.

Les générateurs à radio-éléments. La mise au point de générateurs de radio-éléments (ou " vaches") permettant d'obtenir simplement un radio-isotope de période courte, à partir d'un radio-isotope "père" de période longue, devrait permettre d'éliminer les principaux inconvénients cités pour les radio-isotopes à rayonnement γ.

Les deux couples utilisés jusqu'à ce jour sont l'étain-indium et le césium-baryum. La période de l'indium, voisine de 1 h 30, est très appropriée aux jaugeages en rivière, mais celle de l'étain est malheureusement trop courte (6 mois). La période très réduite du baryum semble à priori limiter son emploi aux rivières où les conditions de bon mélange sont très favorables.

Parmi les générateurs utilisés, celui mis au point par la Direction des études et recherches à l'Electricité de France permet des mesures industrielles en conduite avec le césium-baryum, par la méthode des nuages radio-actifs. A notre connaissance, il n'a pas encore été utilisé pour des jaugeages en rivière par dilution.

Un dernier générateur, conçu par ce même service et pouvant utiliser indifféremment le césium-baryum et l'étain-indium (fig. 3), est actuellement expérimenté par l'Electricité de France sur une rivière des Alpes. Dans ce cas, il met en œuvre la méthode de dilution d'une manière qui s'apparente au procédé par intégration. Cette expérimentation entre dans le cadre de la recherche d'un moyen de contrôle permanent du débit.

V. CONCLUSION

Les méthodes de dilution ont fait des progrès considérables au cours des vingt dernières années. La facilité de leur mise en œuvre, la rapidité des mesures, les améliorations constantes apportées dans les procédés d'analyse ou de comptage et l'arsenal des traceurs possibles, qui ont permis d'étendre l'application de ces méthodes à des débits de plusieurs milliers de m^3/s, devraient permettre d'envisager une extension encore plus grande dans les années à venir.

Grâce à ces procédés, on a pu entreprendre les premières tentatives de contrôle direct et permanent du débit des rivières, sans enregistrement de variables intermédiaires telles que les hauteurs d'eau.

Aux Etats-Unis, dans le Colorado, le professeur Goodell a expérimenté un tel procédé en utilisant une injection continue à débit constant d'une solution de rhodamine Wt : un film 35 mm ne contenant que de la gélatine sans émulsion sensible se déroule à vitesse constante dans une cuve où coule de façon continue de

l'eau prélevée dans une section où le mélange est homogène. La gélatine concentre la rhodamine. Le film est ensuite analysé dans un fluorimètre qui enregistre des valeurs liées directement aux débits de la rivière.

Avec une cuve de solution d'un volume voisin de 100 litres l'autonomie du système est de l'ordre de la semaine. La précision semble comprise entre 5 et 10 %. Quelques difficultés subsistent pour assurer la constance du débit très faible d'injection.

Les résultats des premiers essais effectués en France par utilisation d'un générateur étain-indium, en injectant à des intervalles de temps constants une activité d'indium connue et en comptant à l'aval à l'aide d'une sonde à scintillation immergée dans la rivière, montrent qu'une sensibilité satisfaisante peut être obtenue (utilisation d'une cartouche de 200 mCi d'étain pour les débits inférieurs à 40 m^3/s). Ils ont permis de commencer à mettre en évidence l'influence, sur la mesure, du bruit de fond, de la variation de la géométrie de comptage . . . Les difficultés rencontrées jusqu'à ce jour concernent plus particulièrement la constance de l'activité injectée périodiquement.

Des années de mise au point seront certainement nécessaires pour permettre l'utilisation pratique de ce procédé dans des conditions économiques acceptables, mais à l'heure actuelle aucune des difficultés rencontrées n'apparaît insurmontable.

En cas de succès, le mode d'exploitation d'un réseau de stations de jaugeages traditionnel serait profondément transformé par la suppression de la plus grande partie des jaugeages nécessaires au tarage des stations et du processus de transformation des enregistrements de hauteurs en débits.

DISCUSSION

Keller: Having heard three very interesting papers on dye dilution techniques, I should like to raise the question of pollution—a problem of increasing importance. Does any of the authors know about some studies performed on the influence of sodium-bichromate on biology or fish life in streams?

André: Des études ont été faites, à ma connaissance, en France et au Royaume-Uni. Les concentrations que nous utilisons sont très inférieures au mg par litre et ne mettent pas en danger la vie des poissons. Un poisson pourrait vivre pendant des années dans une solution à 1 mg par litre. Seules les daphnées, petites plantes servant à la nourriture des truites, sont touchées, elle meurent au bout de six jours lorsqu'on les plonge dans une eau à 1 mg de bichromate par litre. Comme nous ne faisons des mesures que de façon très espacée, pendant une heure seulement, et que la dilution augmente afin de diminuer la concentration à l'aval, je pense que nous ne prenons absolument aucun risque.

Techniques for measurement of discharge by dye dilution

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Abstract. Dye-dilution type discharge measurements have been made by the United States Geological Survey for a wide variety of conditions to ascertain the suitability of the method for flow conditions where the current meter is not entirely adequate. The development of a reliable pneumatically-powered constant-rate injection apparatus and of stable fluorescent dyes measurable by fluorometers at concentrations of less than 0.1 μg/l enhances the constant-injection plateau method of dilution gauging. This method has been used extensively in gauging turbulent mountain streams, flow beneath ice, flow in shifting sand-channel streams, and in numerous canals and man-made structures.

TECHNIQUES DE MESURE DU DEBIT PAR LA DILUTION DE COLORANTS

Résumé. Cet article décrit les procédés pratiques, les colorants et les instruments utilisés par l'United States Geological Survey pour mesurer les débits par la méthode de dilution des colorants dans un grand nombre de cas où les méthodes habituelles ne sont pas tout à fait appropriées.

Le développement de colorants fluorescents stables et de fluoromètres capables de mesurer avec précision les faibles concentrations a ravivé l'intérêt pour les méthodes utilisant la dilution pour la mesure du débit.

Les injections à vitesse constante ont fait que les méthodes d'injection continue de concentrations sont devenues préférables aux méthodes dites d'injection lente et de récupération totale. Les systèmes d'injection multiples et les techniques d'échantillonnage ont rendu possible la mesure des écoulements par les méthodes de dilution.

Cette technique est particulièrement appropriée pour la mesure de l'écoulement turbulent et de faible profondeur dans les ruisseaux de montagne, sous une couche de glace, dans des chenaux de sable, dans de nombreux canaux et dans d'autres ouvrages réalisés par l'homme.

TÉCNICAS PARA LA MEDICIÓN DE CAUDAL POR DILUCIÓN DE COLORANTES

Resumen. El U.S. Geological Survey ha realizado mediciones de caudal por dilución de colorantes dentro de una amplia variedad de condiciones, a fin de averiguar la adaptación del método para corrientes en las cuales el medidor de corriente no es enteramente adecuado. El desarrollo de un aparato accionado neumáticamente para inyección a velocidad constante, de garantía y apto para colorantes fluorescentes estables medibles por fluorómetros con concentraciones de menos de 0,1 μg/l, favorece el método de inyección constante para aforo por dilución. Este método se ha utilizado ampliamente para medidas en cursos de agua turbulentos de montaña, corrientes bajo hielo, corrientes en cursos arenosos, y en numerosos canales y estructuras artificiales.

ОБОРУДОВАНИЕ ДЛЯ ИЗМЕРЕНИЯ РАСХОДА МЕТОДОМ РАЗБАВЛЕНИЯ КРАСКИ

Резюме. Измерения расхода с помощью разбавления краски выполнено Геологической Службой США для широкоменяющихся условий с целью выяснения пригодности этого метода для условий, где вертушка совсем неприменима. Разработка надежного прибора, обеспечивающего выпуск в лоток красителя с постоянной интенсивностью (с применением пневмати-

ческого устройства), и устойчивых флюоресцентных красок, измеряемых флюориметрами при концентрациях менее 0,1 мг/л, повышает надежность этого метода. Он был широко использован при измерении турбулентных горных потоков, потоков под ледяным покровом, потоков с деформируемыми (песчаными) руслами и на многих каналах и искусственных сооружениях.

INTRODUCTION

In recent years the United States Geological Survey has been active in the development of dye-dilution gauging techniques. The availability of stable fluorescent dyes and fluoromoters which can accurately measure concentrations of the order of 0.1 μg/l has enhanced the use of dilution methods. In general,dye-dilution gauging of natural streams is not economically competitive with the current meter. However, for extremely turbulent flow or where flow geometry is undefinable or constantly changing, dilution methods have considerable merit.

Tracer gauging may be divided into two categories: (1) the tracer-velocitymethod, and (2) the tracer-dilution method. The former is most commonly thought of as the salt-velocity method, although any tracer may be employed. It has the advantage of simple injection, requires a relatively small amount of tracer, and is not critically dependent on the conservativeness of the tracer. Its chief disadvantages are the requirement that the channel-flow geometry be defined exactly, sampling is extensive, and cloud-to-cloud measurements are required.

Two tracer-dilution methods are available, both based on the fact that mixing a known amount of soluble tracer with the total flow of a stream yields concentrations inversely proportional to the discharge. The chief advantage of dilution gauging is that precise knowledge of the flow geometry is not required.

TRACER-DILUTION METHODS

Slug-injection total-recovery method. This method employs the slug injection of a measured quantity of tracer into the flow and subsequent sampling of the entire time-concentration curve downstream where mixing is adequate. The discharge may then be computed as

$$Q = \frac{V_I \; C_I}{\int_0^\infty c \; dt} \tag{1}$$

where V_I and C_I are the volume and concentration, respectively, of the dye injected, and the denominator is the area of the time-concentraton curve. The advantages of the method are ease of injection and minimum dye requirements. However, as indicated in Figure 1 top, the total mass of dye must be accounted for and unless complete mixing is known to exist, the entire dye cloud must be sampled at 3 to 5 points in the cross-section. Although the slug-injection method is simple and economical, the sampling and analysis are rigorous.

Constant-injection plateau method. In this method a tracer of concentration C_I is injected at a constant rate, q, until an equilibrium or plateau concentration, C_p, is

reached at the sampling point downstream where mixing is adequate. The discharge is then determined as

$$Q = q \frac{C_I}{C_p}. \tag{2}$$

The constant-injection plateau method can be visualized (Fig. 1 bottom) as the integration of the time-concentration curves for a series of closely spaced slug-type injections. The required period of constant injection must equal or exceed the passage time for a slug-injected dye cloud, ΔT_c, at the sampling section to achieve the plateau concentration. The advantages of the constant-injection method are:

1. Only a few samples are necessary to determine the plateau concentration, C_p.
2. Samples may be collected any time during the steady state-plateau concentration period.
3. From a practical sense, generally one man can make a measurement, operating the injection apparatus and collecting samples down-stream.
4. Sampling the entire flow section provides a measure of the adequacy of mixing, hence improving the accuracy and reliability of the measurement.

The chief disadvantages are the requirement of a constant flow-rate apparatus, and the greater quantity of tracer required to measure a given discharge.

MEASUREMENT REACH

The channel reach between the points of injection and sampling must be such that the tracer is adequately mixed with the flow. The dye-cloud passage characteristics of slug and constant-rate injections with sampling reach length are illustrated in Figure 2, using concentration-time curves. The optimum reach is the shortest length of channel that produces adequate mixing. Practical experience indicates the optimum mixing length, L_m, may be estimated by the formula

$$L_m \approx KV \frac{W^2}{D} \tag{3}$$

where V is the mean velocity in metres or feet per second; W is the mean width of the stream in metres or feet; D is the mean depth of the stream in metres or feet; and K is a coefficient depending on the method of injection and type of channel and flow conditions as suggested in Table 1.

Table 1. *Mixing length coefficients*

Type of injection	Channel and flow conditions	Coefficient, k	
		(sec/m)	(sec/ft)
Single, bankside	Straight and uniform	8.5	2.6
Single, midchannel	Straight and uniform	4.9	1.5
Single, midchannel	Pool and riffle	3.3	1.0
Multiple, 3 to 5 points	Straight and uniform	1.6 to 1.0	0.5 to 0.3

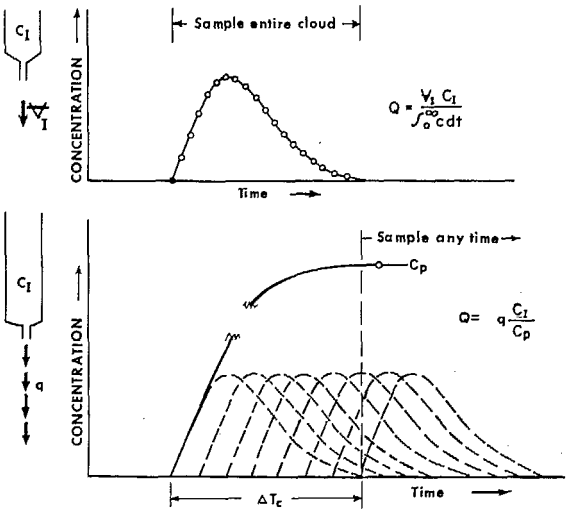


Figure 1. *Dye-dilution discharge measurements by slug and constant rate injection methods*

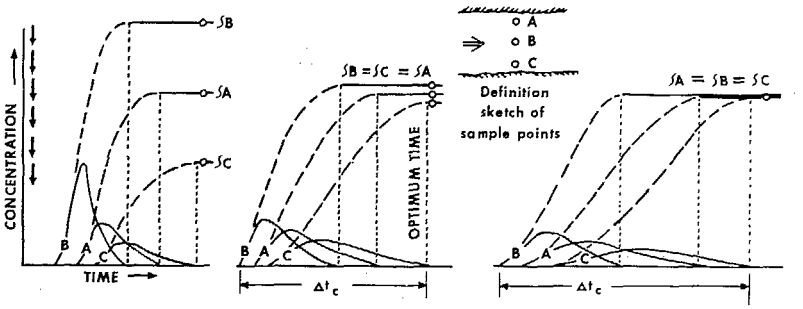


Figure 2. *Slug and constant injections, time-concentration curves to be observed at three points laterally across channel at three different distances below the injection point*

The shortest mixing lengths are usually found on pool and riffle streams because the pools serve as mixing basins.

Generally, the excessive mixing length becomes a practical limitation of dilution gauging rather than discharge and dye quantities. The use of multiple injections greatly reduces the optimum mixing length, as can be seen in Table 1. A simple manifold attachment to the injection line from the dye tank makes multiple injections possible.

A site where routine constant-rate dye-dilution measurements are to be made should be calibrated for optimum mixing lengths, injection periods, and sampling times for a range in discharges. Perfect mixing generally is not necessary and usually requires excessive channel lengths.

CONSTANT-RATE INJECTION APPARATUS

The application of the constant-injection method requires a reliable apparatus capable of injecting a dye solution at a constant rate. Figures 3 and 4 show an

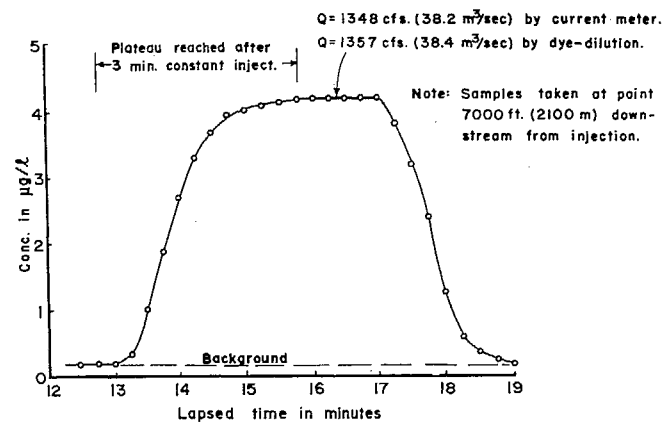


Figure 3. *Rapid measurement of discharge in a lined canal by dye-dilution, Charles Hansen Canal, Colorado*

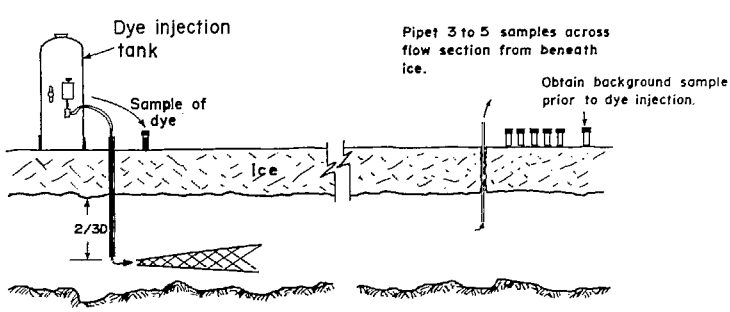


Figure 4. *Dye-dilution discharge measurement beneath ice cover*

adaptation of a pneumatically-powered chlorine-feed tank manufactured by Aero-feed, Inc., Chalfont, Pa. A slight alteration of the orifice control unit provides a range of constant-flow-rate settings from 0.5 to 4 ml/sec, accurate within 1 per cent. Flow is regulated by means of a spring-diaphragm mechanism which maintains a constant differential pressure across an adjustable orifice regardless of changes in tank or external pressures. The unit is loaded with 8 litres of dye solution, sealed, and pressurized with an air pump. It weighs approximately 25 pounds (11 kg) when full, and is readily portable. This unit, with a dye solution concentration of 2×10^7 μg/l (2 per cent), can be used to measure discharges up to 700 cubic feet per second (cfs; equivalent to 20 m^3/sec). With greater dye-solution concentrations, proportionally larger discharges may be measured.

LABORATORY ANALYSIS

These dye dilution techniques are capable of accuracies equal to current-meter measurements under favourable conditions. This is due in part to the quality of fluorescent dyes and fluorometers now available. Rhodamine WT is the dye most commonly used because it is a more conservative tracer than most other dyes presently available. The Turner 111 filter-type fluorometer, with suitable accessories and proper laboratory techniques, is capable of measuring dye concentrations within 1 per cent.

In practice, a supply solution of dye is prepared of such concentration as to measure the anticipated range of discharges. For calibration of the fluorometer, a sample of this supply solution is reduced by a serial dilution process to concentrations covering the range that will be obtained in the field measurements. Once calibration standards have been prepared from the supply solution, the fluorometer analysis of these and the half-dozen field samples comprising a dilution-type discharge measurement, requires only minutes.

APPLICATIONS

Turbulent mountain-streams. The constant-rate dye-dilution method is particularly useful in measuring turbulent mountain streams at flows either too small and shallow, or too large and turbulent for reliable current-meter measurements. A measurement in a reach of Owl Creek, a mountain stream in Wyoming, consisted of five samples collected across the channel at a point 800 ft (244 m) downstream from a midchannel injection of about 15 minutes' duration. The injection rate of about 1.5 ml/sec was determined volumetrically using a 100-ml graduated cylinder and stopwatch.

Conduits canals, and man-made structures. The method is suitable for measuring in closed conduits, canals, and at man-made structures. Seven measurements, ranging from 0.5 to 45 cfs (0.014 to 1.27 m^3/sec) were made with one tank of dye. The injection rate is measured using a laboratory balance and stopwatch. Weighing is frequently used to measure the injection rate for field measurements, especially for multiple injections since only the total rate is desired.

Figure 3 shows the time-concentration curve and results of a 5-minute injection, 7,000 ft (21 m) upstream from the sampling point.

Ice-covered streams. The measurement of flow beneath ice-covered channels (Fig. 4) is greatly simplified by using the constant-rate injection technique. Dye is injected through a small hole in the ice. The injection line outlet is supported clear of the underside of the ice to avoid "painting". Samples are obtained by pipetting through several small holes in the ice. Methanol alcohol or glycol added to the dye solution eliminates freezing problems.

Sand-channel streams. Many sand-channel streams in the western United States are characterized by a constantly changing bed configuration and fluctuating flow conditions. These conditions can have an adverse effect on the accuracy of current-meter measurements, but have little effect on the accuracy of dye-dilution measurements. Figure 5 A and 5B show the variations in depth and discharge for three current-meter measurements made during and 8-hour period on the Middle Loup River at Dunning, Nebr.

Sand-channel streams are usually wide relative to depth, and require very long mixing lengths for single-point injections. The effect of multi-point injection in

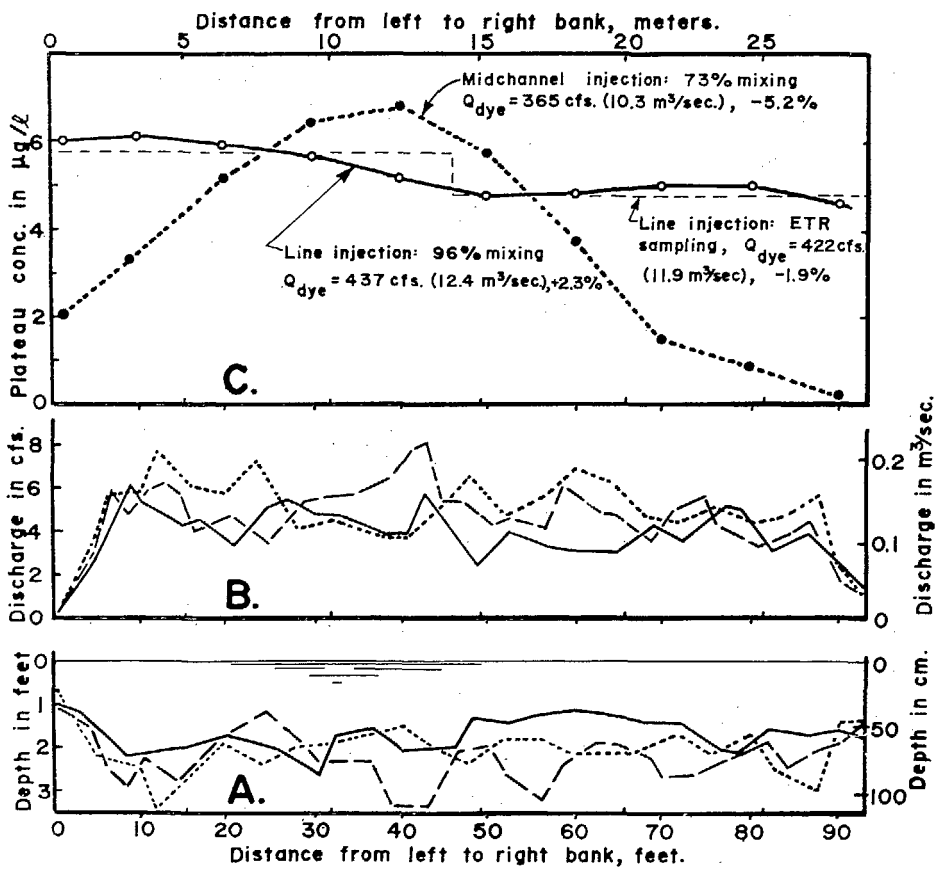


Figure 5. *Variation of depth, discharge and dye concentration for single-and multi-point injections with width during an 8-hour period on the Middle Loup River, a sand-channel stream in Nebraska*

reducing the optimum mixing length is illustrated in Figure 5C. The lateral distribution of the plateau dye-concentration for single and multi-point injections is compared at a relatively short mixing length of 100 ft (33.5 mètres). The optimum mixing length for a single-point injection would be about 12,000 ft (3,660 m).

Equal Transit Rate (ETR) sampling, the conventional method of sediment sampling, can be used to collect integrated samples of the dye concentration, as well as sample-suspended sediment. This technique is possible only with the constant-injection method, as a steady plateau concentration must exist during the sampling. Because this method of sampling discharge weights the dye concentration, good results can be obtained with poorer mixing and hence shorter channel reaches.

Automatic and semi-automatic dye-dilution gauging stations. While accurate dilution-type discharge measurement can be made on the types of natural streams thus far discussed, it is frequently difficult to establish stable stage-discharge relationships. Thus, the continuous or periodic measurement of flow and computation of stream-flow record independent of stage would be most desirable. While still in the experimental stage, the feasibility of operating automatic dye injection and sampling equipment is encouraging. Such gauging stations will probably take two forms: (1) the completely automated dye injection and sampling system, and (2) the semi-automatic station comprised of permanently installed, but manually operated, injection and sampling equipment.

DISCUSSION

André: 1. La rhodamine n'est-elle pas retenue dans les sables et les argiles ?

2. La formule $L_m \approx KVW^2$ donnerait pour le Betsiboka une longueur de bon mélange de plus de 1 000 km alors qu'il a suffi de 3 km ; ma formule donne également de très mauvais résultats et je pense que toutes ces formules ne peuvent pas être universelles et sont dangereuses.

3. Des essais effectués par nous-mêmes et par nos collègues du Royaume-Uni ont montré que pour un mélange à près de 10 % les injections multiples réduisent notablement la distance de bon mélange, alors que pour un mélange à près de 1 % il n'y a pas de différence notable, que l'injection se fasse en un ou en plusieurs points.

Barnes: 1. We are aware that it is dangerous to use the above formula and we do not recommend it except as a guide-line to establish the calibration. In other words, the formula is not used. We recommmend that the sampling points, time of injection, injection rate, and the time to collect samples be based on calibration measurements.

2. Suspended matter has not been a great problem with rhodamine WT. Successful measurements have been made with a concentration of the order of several 1,000 ppm.

3. Third question: we normally do not recommend more than a 5-point injection because this leaves a very small return for the extra injection.

Liu: I too feel that equation 3 can be correct only within a certain range of conditions. It appears to me that for a very large width to-depth ratio, the "mixing length", L_m, is mainly determined by the width of the flow rather than the width. From the form of equation 3, it seems that the equation could only be correct for

wide-open channels, within a certain range of width-to-depth ratio. I should like to know whether the equation was actually derived from experimental data taken in wide-open channels, and whether the range of conditions under which equation 3 holds true could be specified.

Barnes: The coefficients have been developed on the basis of those experimental measurements. The values of width and depth are approximate, since we only measure one or two points in a reach. These coefficients reflect any errors and estimates on the estimated parameters. The range of applicability of the formula is related to the channels used. The greatest width applied was more than 100 feet (30 m) ; the greatest depth was of the order of 10 feet (3 m).

Sauzay: Dans le texte, les auteurs utilisent la même phrase "where mixing is adequate" pour la discussion des sections de mesure, en ce qui concerne les deux méthodes de dilution. Il est nécessaire de rappeler que ceci recouvre deux notions différentes en théorie :

1. Méthode de l'injection instantanée : le bon mélange nécessaire à l'application de la formule 1 n'est pas représenté par une concentration uniforme dans la section de mesure, mais par une valeur constante de l'intégrale *cdt.*

2. Méthode de l'injection continue : la concentration doit être uniforme sur toute la section de mesure.

Ces deux conditions de "bon mélange" sont différentes.

Barnes: I agree that the problem is to compare the two mixing conditions. The way of mixing determines the degree of accuracy of the measurement. When the mixing is of the order of 95 per cent, the comparisons with current-meter measurements are of the order of 1-1.5 per cent.

Stream hydrographs by fluorescent tracers

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Abstract. A continuous stream-gauging system is described, whereby fluorescent tracers are used to produce stream hydrographs without the supplementary recording of stage. In the system, a tracer solution of known concentration is constantly injected at a known rate into the stream. Downstream, where the tracer is well mixed, sample flows are diverted through a self-operative instrument which registers tracer concentrations with time on a gelatin-coated flim. An expensive, temperature-sensitive fluorometer need not be stationed at the stream side. The "exposed" film is periodically gathered from stream sites, stored if necessary, and analysed in a laboratory-based fluorometer.

Hydrographs resulting from 640 hours of gauging of two Colorado mountain streams with the system are compared with those obtained from closely located sharp-crested weirs. The maximum instantaneous deviation between hydrographs reaches 10 per cent and the average absolute departure equals 1.8 per cent, while algebraic departure averages + 0.3 per cent.

HYDROGRAMMES OBTENUS PAR L'EMPLOI DES TRACEURS FLUORESCENTS

Résumé. Dans cette note on décrit un moyen de mesurer de façon continue le débit d'un cours d'eau par l'utilisation de traceurs fluorescents pour obtenir des hydrogrammes du cours d'eau sans qu'il soit nécessaire d'enregistrer les hauteurs d'eau. Selon ce système, une solution de traceur d'une concentration donnée est introduite dans le cours d'eau à une vitesse constante prédéterminée. On dispose un appareil pour mesurer la concentration du traceur en aval du point d'introduction du traceur et à une distance suffisante pour être sûr que le traceur soit bien mélangé à l'eau. Cet appareil enregistre automatiquement, sur une pellicule enduite de gélatine, l'heure et la concentration du traceur de divers échantillons d'eau prélevés dans le cours principal. Il n'est donc pas nécessaire de placer à côté du cours d'eau un fluoromètre coûteux et souvent sensible aux variations de température. Les pellicules impressionnées sont recueillies périodiquement sur le terrain, entreposées si nécessaire et analysées au moyen d'un fluoromètre.

Dans cette étude, des hydrogrammes résultant de 640 heures de mesure de débits de deux torrents au Colorado, sont comparés à ceux obtenus de plusieurs déversoirs à crête mince situés à proximité. Le maximum de la déviation instantanée entre les hydrogrammes atteint 10 % ; la moyenne de l'écart absolu est égale à 1,8 %, tandis que la moyenne de variation algébrique est de 0,3 %.

HYDROGRAMAS DE CORRIENTE POR TRAZADORES FLUORESCENTES[1]

Resumen. Se describe un sistema de aforo de corriente continuo, en el cual se utilizan trazadores fluorescentes para obtener hidogramas de corriente sin medidas supletorias de nivel. En el sistema se inyecta una disolución de trazador de concentración conocida, a una velocidad también conocida, en la corriente. Aguas abajo, donde el trazador se encuentra bien mezclado, se separan muestras con un instrumento que registra las concentraciones en función del tiempo en una película cubierta de gelatina. No es necessario situar en la corriente un fluorómetro sensible a la temperatura. La película "expuesta" se recoge periodicamente de los emplazamientos en la corriente, se almacena si es necesario, y se analiza en un fluorómetro situado en el laboratorio.

Los hidrogramas obtenidos en 640 horas de aforo en dos ríos de montaña en Colorado con este sistema se comparan con los obtenidos por medidas directas. La desviación máxima instantánea entre hidrogramas alcanza el 10 % y la separación media absoluta es del 1,8 % mientras que la algebráica en términos de media es del + 0,3 %.

ГИДРОГРАФЫ СТОКА, ПОЛУЧЕННЫЕ С ПОМОЩЬЮ ФЛЮОРЕСЦЕНТНЫХ ИНДИКАТОРОВ

Резюме. Описывается система продолжительного измерения стока, где флюоресцентные индикаторы используются для получения гидрографа стока без дополнительной регистрации уровня. В этой системе раствор индикатора известной концентрации непрерывно вводится с известным расходом в реку. Ниже по течению, где индикатор хорошо перемешан, пробные потоки отводятся через саморегулируемое устройство, которое регистрирует изменение концентрации индикатора во времени на пленке, покрытой желатином. При этом нет необходимости в установке на берегу реки дорогого, весьма чувствительного к изменению температуры флюорометра. « Экспонированная » таким образом пленка периодически собирается, накапливается, если это необходимо, и анализируется в лаборатории с помощью флюорометра.

Гидрографы, полученные после 640 часов измерений на двух горных реках Колорадо с помощью описанной системы, сравнены с гидрографами, полученными с помощью измерений на близ расположенных тонкостенных водосливах. Максимальное мгновенное отклонение между гидрографами достигает 10 %, а среднее абсолютное отклонение равно 1,8 %, тогда как среднее алгебраическое отклонение составляет + 0,3 %.

INTRODUCTION

The gauging of streamflow at chosen instants of time by means of fluorescent tracers has been successfully researched and developed. (Wright and Collings, 1964; Merritt, 1964; Replogle *et al.*, 1966; Goodell *et al.*, 1967; Kilpatrick *et al.*, 1967.)

The fluorescent tracers found most suitable for the gauging of natural streamflows are the xanthene dyes, Rhodamine WT[1] and Pontacyl Brilliant Pink B.[1] These dyes are detectable in low concentrations, adequately stable, non-toxic in normal use, inexpensive, sufficiently water soluble, and only slightly sorbed on channel beds and suspended sediment.

Most favoured in such gauging has been the constant-injection technique in which a tracer solution of known concentration is continuously discharged into the stream at a steady, pre-set rate and the dosed stream is sampled at a downstream point where mixing has become complete. Under the condition of steady flow during the travel time of the tracer, a single sample of the dosed water enables calculation of the discharge rate of the stream.

The equation pertinent to the calculation is:

$$Q = q \frac{C_1 - C_2}{C_2 - C_0} \qquad (1)$$

where Q is instantaneous rate of stream discharge, q is rate of injection into the stream of a tracer solution of concentration C_1, C_2 is tracer concentration in the stream after complete mixing, C_0 is the naturally occurring concentration of the tracer and tracer-like substances in the stream.

1. Product of E.I. DuPont de Nemours Company, Willmington, Delaware, U.S.A.

In practice, when suitable tracers are employed, C_2 is negligibly small relative to C_1. Similarly, C_0 is commonly zero. Thus, équation 1 may be simplified to

$$Q = q \frac{C_1}{C_2}. \tag{2}$$

With appropriate equipment and procedure we can keep q and C_1 constant over any chosen time period. Then

$$Q = K \frac{1}{C_2}. \tag{3}$$

If C_2 were sampled continuously, stream discharge could be evaluated by equation 3, to yield a stream hydrograph without the supplementary recording of stage.

Applications of continuous gauging by this tracer-dilution technique vary. Turbulent, tortuous mountain streams are inimical to accurate use of the current meter, but favour the tracer technique because of their potential for rapid mixing. The technique can also be used to produce a hydrograph on streams whose bed configuration varies excessively with flow rate, rendering its stage-discharge relation invalid.

Continuous gauging by tracer-dilution is currently possible with commercially available equipment. A fluorometer, with recorder, pump, and power source may be appropriately sited on the stream, together with means for the continuous injection of a dye solution, to yield a continuous record that can be converted to a hydrograph of discharge. However, there are physical and economic objections to placement of the fluorometer at the stream site. Physical objections originate from the temperature dependence of fluorometer performance (Dunn and Vaupel, 1965) and from the difficulty of providing an adequate power source at a remote site. Economic objections arise from the on-site capital investment in instrumentation and from the logistics of protecting the latter.

This paper describes an alternative system for the continuous monitoring of tracer concentrations in a continuously dosed stream and for converting the resultant record to a hydrograph of discharge. It also presents results from fairly extensive tests. The system features low on-site investment, with ready portability, and has given accurate hydrograph delineation under the test conditions encountered. Although not recounted in this paper its utility in time-of-travel and dispersion studies has also been established.

FIELD EQUIPMENT AND PROCEDURES

Field equipment of this system is basically very simple, but required considerable experimentation before it operated satisfactorily. Reliability with infrequent attention was mandatory. Simplicity, minimum bulk, and limited dependence on electrical power were also considered to be essentials.

For the constant discharge of the tracer solution into the stream we adopted a tank designed and developed by Goodell *et al.* (1967), and modified by Siverts (1967). Based on the principle of the Mariotte vessel, this tank can discharge a

steady stream of tracer solution at a rate of 4 ml per minute over a week-long period.

The equipment for continuously monitoring the tracer concentration (C_2) in the dosed stream consists of two major components: a stream sampler and a tracer registration device with battery pack.

The sampler was designed to provide continuously a small, steady aliquot of the dosed stream to the registration device (Fig. 1). A screened intake vessel anchored to the stream channel bottom allows gravitational flow of about 10 litres per minute through a hose to a stream-side, open-topped tank of about 28 - litre capacity. Free overflow from this tank maintains a constant head on a hose leading to the tracer registration device.

The tracer registration device (film-dye contactor of Fig. 1) performs the central task of continously recording the tracer concentration in the stream aliquot. Flow from the constant head tank supplies an impoundment of about 155 cm^3 in a reservoir within the registration device. Liquid level in the reservoir is maintained constant by a standpipe drain through which the flow returns to the stream.

Immersed in the impounded flow to a pre-set and constant depth is a loop of gelatin-coated film.[1] The film, which is perforated for sprocket gear drive, is drawn through the tranquil flow of the stream aliquot in the reservoir by a timing motor powered by a 30-volt battery pack. From the reservoir, the film passes through a drying chamber activated by a saturated aqueous solution of magnesium chloride and then into storage on a motor-driven spool. At periodic intervals, such as a week, the "exposed" length of film may be removed for transport to the laboratory and analysed. The complete field assemblage, injector tank, stream sampler, and tracer registration device, is diagrammed in Figure 2.

The film we have used has a polyester base and a single-side gelatin coating of 20 micron thickness. It is 35 mm wide and is available in 30 m lengths. The registration of dye concentration takes place within the gelatin-coating. Upon contact with the dye-dosed stream water, the swelling gelatin will absorb and bind dye molecules into its matrix. Although the total process involved is complex, the quantity of dye taken up by a unit area of gelatin is directly and linearly proportional to dye concentration in the stream aliquot, if other influential factors are constant.

Of the additional factors that could be effective on dye uptake, the only ones found to be consequential were: (1) contact time between film and stream aliquot, (2) temperature of the aliquot, and (3) amount of suspended sediment in the sample.

We found that uptake of Rhodamine WT is an exponential function of contact time such that little takes place beyond 60 minutes and 60 per cent of the 60 minute quantity is accomplished in 19 minutes. On this evidence, we chose 19 minutes as our nominal exposure time in field trials. We secured this exposure by maintaining a film speed of 11.43 cm per hour and a film immersion length of 3.62 cm. Continouously registered values of C_2 represent the integration of C_2 over 19 minutes and thus some smoothing of temporal variations in C_2 is inherent in the record.

Laboratory tests indicate that within the range, 7°-35°C, the uptake of Rhodamine WT increases 0.4 per cent per °C. temperature rise; outside these limits

1. Dry Bimat Tranfer Film, Type 2436 - A ; Product of Eastman Kodak Company Rochester, New York, U.S.A.

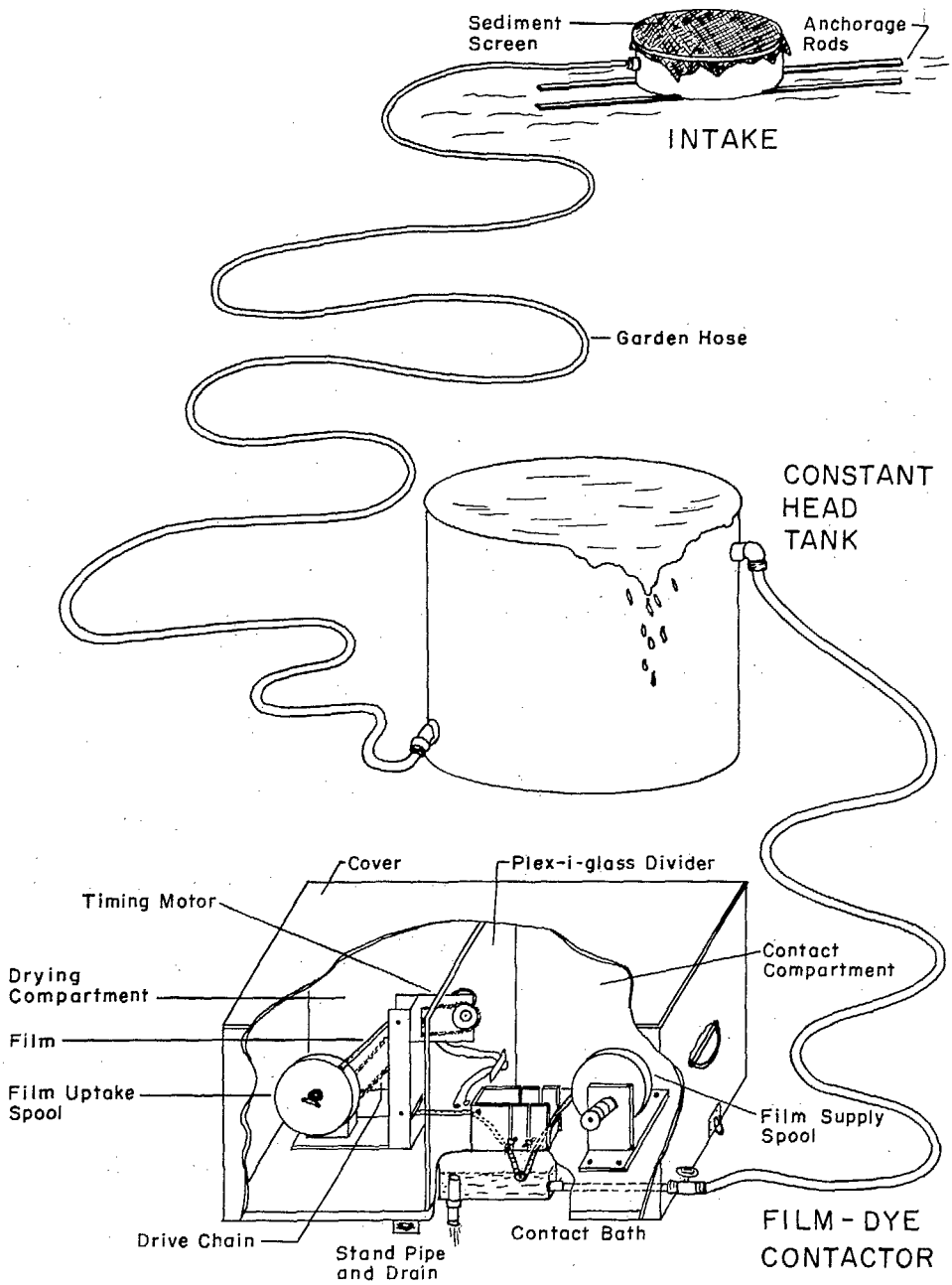


Figure 1. *Stream-side tracer registration device, or film-dye contactor, shown with flow route of the stream aliquot*

response increases slightly. We believe that, in general, fluctuations in stream water temperatures within weekly periods are sufficiently small that their effects on C_2 registration and, in turn, on the accuracy of hydrograph delineation can be ignored.

Suspended sediment in streams presents no greater problems to continuous gauging by dye dilution than it does to instantaneous gauging. We tested our gauging system with a 12 ppb (parts per U.S. billion) Rhodamine WT solution containing 2,600 ppm concentration of sediment consisting of 76 per cent clay, 19 per cent silt, and 5 per cent fine sand. On routing this continuously agitated mix through the reservoir of the tracer registration device for two hours, we found that the quantity of dye taken up was 2.5 per cent less than from a comparable solution free of suspended sediment. Replogle *et al.* (1966) tested the sorptive potential of similar sediment for the same dye. Using a dye strength of 10 ppb, a suspended sediment concentration of 1,000 ppm and measuring the fluorescence of liquid samples, they found a 6 per cent dye loss by sorption.

LABORATORY EQUIPMENT AND PROCEDURES

One laboratory facility can easily accommodate the analyses of the exposed film from 15 or more stream-side installations. Exposed film can be stored in the laboratory for a year or more, if necessary, or it can be fluorometrically analysed immediately.

Figure 3 pictures the two major components of the laboratory facility: a modified fluorometer and a strip-chart, milliampere recorder. The fluorometer[1] is the null-balancing type featuring a servo-system. We modified the sample-holding door of this instrument to accommodate a strip of gelatin-coated film so that it could be drawn by clockwork across a diffusely reflected surface for automatic scanning. We used a film speed of 11.43 cm per minute. At any given instant, the fluorometer, as modified, responds to the integrated fluorescence from a 3.4 cm^2 area of film.

The fluorescence of Rhodamine WT is inversely proportional to temperature. Experience showed that ambient temperature is influential on fluorometer performance. To take advantage of the first fact and minimize errors caused by the second, we provided a water-cooling system for the reflective surface against which the film is impinged during analysis and also took care that the fluorometer was subject to minimal temperature fluctuations during use.

The fluorescence from the film is a linear function of the dye content and thus of the C_2 of the stream water. The output of the fluorometer is in direct current millivolts or milliamperes. The record on strip-chart or computer-acceptable tape represents a continuous time series of C_2 values or, by polarity reversal, represents $1/C_2$.

Conversion of the time-dependent tracer concentrations into the stream hydrograph requires at least one, preferably two or more, time-noted instantaneous measurements of stream discharge. Normally, these measurements would be made by means of discrete samples of the dosed streamflow taken at periodic visits to the gauging site for servicing.

1. Turner, Model 111, Fluorometer ; manufactured by G. K. Turner Associates, Palo Alto, California, U.S.A.

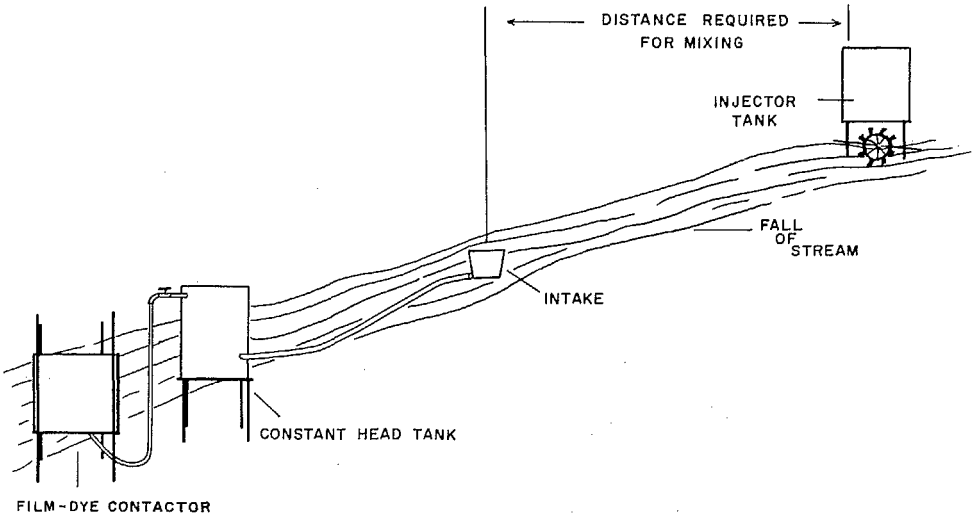


Figure 2. *Dye injector is mounted over stream or on stream bank. At a downstream point where mixing is complete, a small flow is hose-diverted through a constant head tank and the film-dye contactor*

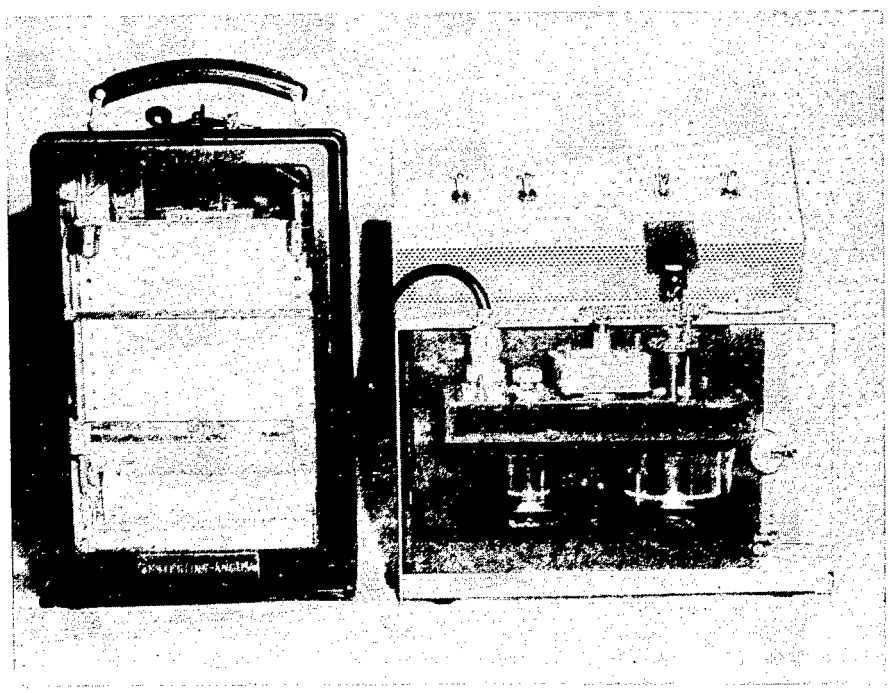


Figure 3. *Fluorometer with modification to accommodate continuous strip of film under clockwork control and a continuous strip-chart recorder*

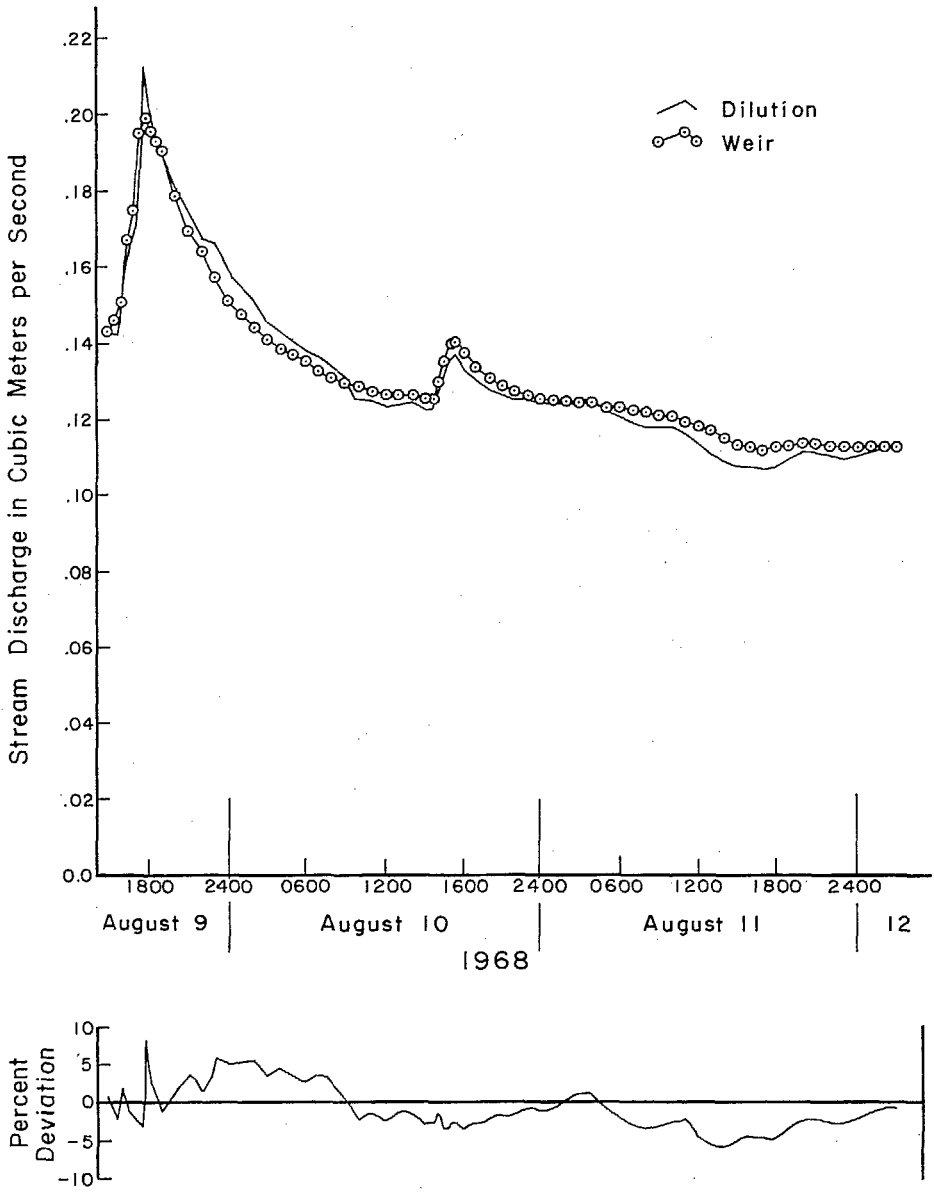


Figure 4. *Hydrograph comparison, gauging period 3, East Saint Louis Creek*

Equation 3 relates the discharge Q, to the reciprocal of dye concentration, $1/C_2$, by the proportionality factor K. For each gauging period, K is invariant except for any random variations inherent in the gauging system. Thus, K for a period (week) is obtained by averaging the K values obtained from each instantaneous measurement of discharge and the film-recorded C_2 value corresponding in time. Application of this average K to the recorded time series of $1/C_2$ yields the stream hydrograph.

FIELD TRIALS

Field trials of the gauging system were conducted in 1967 and 1968 on two mountain streams, East Saint Louis and Deadhorse Creek, within the Fraser Experimental Forest in the United States. Test sites were chosen near existing sharp-crested weirs to allow inter-system comparisons. Sufficient Rhodamine WT was used to maintain C_2 concentrations within a total range of 2-14 ppb, but more commonly within 6-8 ppb.

Because instrument and technique development was contemporaneous with many of the field trials, recognizable failures in performance were numerous. Data from time periods encompassing such failures were excluded from data to be statistically analysed. There remained 640 hours of record from 18 discrete gauging periods that were free of known and vitiating faults.

Weir-based discharge data from the beginning and ending of each period were used to derive K values for the conversion of $1/C_2$ data to Q values. The congruity of the tracer-dilution-based and weir-based hydrographs within each of the 18 periods was then studied. Figure 4 shows the congruity for one gauging period (No. 3). Table 1 presents a summary of comparisons within all 18 gauging periods. Although an instantaneous maximum of 10 per cent occurred, tracer-dilution-based measurements differed from weir-based measurements by an algebraic average of only 0.32 per cent of the latter and by an absolute average of only 1.83 per cent.

Results by the two systems were also compared by a regression analysis based on 916 pairs of streamflow values taken from the 18 periods. To hourly values were added values at smaller intervals of time when discharge change was rapid. The analysis showed that stream discharge determinations by the two systems are in a near-perfect regression slope agreement of 1.00059. At a discharge of 0.20 m^3/sec, where confidence interval is at a maximum, the value of the latter at the 1 per cent is $\pm$ 0.00047 m^3/sec.

CONCLUSION

The stream gauging system herein reported performed well during experimental trials on mountain streams. The utility of the basic concepts seems well established. Further trials and refinements should be made to optimize convenience and reliability, to minimize dye requirements, and to determine its adaptability to other types of streams.

It is not expected that the system will supplant gauging by means of control sections where the requirements are for highly accurate, long-term records, and full details of stream fluctuations. Envisaged is particular utility in water resource inventory by a core of conventional, permanent gauging stations, each one having a number of roving or "satellite" stations of the type described here.

Table 1. *Summary of hydrograph comparisons, Deadhorse and East Saint Louis Creeks*

Gauging period	Duration hours	Deviation between tracer-dilution and weir systems		
		Instantaneous maximum (%)	*Averages*[1] Absolute (%)	Algebraic (%)
Deadhorse Creek				
1	39	— 10.0	2.29	0.34
East Saint Louis Creek				
2	34	— 2.5	0.69	— 0.23
3	60	8.2	2.66	0.78
4	40	± 3.5	1.76	— 0.24
5	17	6.8	2.45	0.80
6	19	— 6.0	2.31	— 2.10
7	10	± 0.5	0.19	0.01
8	69	6.5	2.84	1.94
9	25	4.9	2.48	1.96
10	27	± 2.1	1.43	1.25
11	23	— 4.2	1.14	— 1.01
12	38	— 5.4	1.93	— 0.78
13	65	— 8.9	1.43	0.31
14	14	2.8	0.86	0.48
15	38	2.5	1.12	0.01
16	53	4.0	1.69	1.54
17	24	— 5.8	2.26	— 1.92
18	45	— 4.4	1.09	— 0.61
Grand average	640		1.83	0.32

1. Based on deviations at hourly intervals or less.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Dunn, B.; Vaupel, D. E. 1965. Effects of sample and fluorometer compartment temperatures on fluorometer reading. *U.S. Geological Survey Professional Paper 525-D,* p. D225-7.

Goodell, B. C.; Watt, J. P. C.; Zorich, T. M. 1967. Streamflow volumes and hydrographs by fluorescent dyes. *14th Congress of International Union of Forestry Research Organization,* section 01-02-11.

Kilpatrick, F. A.; Sayre W. W.; Richardson, E. V. 1967. Flow measurements with fluorescent tracers (Discussion). *Journal of Hydraulic Division, American Society of Civil Engineers,* vol. 93 (HY4), p. 298-308.

Merritt, W. F. 1964. A study of the dilution in the Ottawa River using Rhodamine B. *Health Physics,* vol. 10, p. 195-201.

Replogle, J. A.; Myers, L. E.; Brust, K. J. 1966. Flow measurements with fluorescent tracers. *Journal of Hydraulic Division, American Society of Civil Engineers,* Vol. 92 (HY5), Paper 4895, 15 p.

Siverts, L. E. 1967. Application of the dye dilution technique to periodic summations of streamflow from mountain watersheds. Unpublished thesis, Colorado State University, Fort Collins, Colorado, U.S.A. 111 p.

Wright, R. E. ; Collings, M. R. 1964. Application of fluorescent tracing techniques to hydrologic studies. *American Water Works Association Journal,* vol. 56, p. 748-54.

DISCUSSION

Douillet: Le débit d'injection de la solution concentrée était très faible. Vous avez probablement rencontré des difficultés importantes pour réaliser la constance du débit. Quel dispositif avez-vous utilisé ?

Steppuhn: Professeur Goodell has worked on this problem. We found it necessary to use a small paddle-wheel. Later we used the energy from the turbulence of the stream, with the help of a long rod in the stream. Its vibration maintained this low-rate injection.

Faber: Do you use the gelatin of the film as an absorbent for the fluorescence, develop it, and measure the gradation ?

Steppuhn: I describe the process mainly because of its complexity. It involves at least three processes: swelling up the gelatin, diffusion process of the tracer molecules into the gelatin matrix after and during swelling, and then an absorption process, partially chemical, but mostly physical, out of the gelatin precipitate.

Behrens: When measuring the discharge with Rhodamine WT in mountainous areas we have found that a constant rate of discharge of the inidcator by constant injection will not be obtained if the run-off changes rapidly. This however is typical for these areas. In the case of increasing discharge (and increasing velocity) too much indicator flows away and vice versa. This implies mistakes in the measurement. We have found a simple method to correct this mistake by using not only the dilution but also the time interval of run-off. Did you also have this experience with rapidly changing discharge? Further, you are using the Mariotte bottle for a continuous injection over eight days. We have found a strong influence of temperature. How do you avoid changes of temperature during the day ?

Steppuhn: The system is only applicable when a relatively gradual rate of tracer discharge change occurs in the mixing reach. Professor Goodell simply used the cooling potential of the stream itself.

Behrens: We have applied similar methods. A third remark: you mention that fixing the apparatus in one place is a disadvantage. In order to control the quantity of mixed indicator, we have estabished five intakes distributed over the profile which feed the same counter in turn.

Goodell: In response to the earlier question as to difficulties in achieving reliable rates of discharge, I wish to state that we did experience many difficulties. These are recounted in an earlier paper, given as the second reference in the present paper. I should like to ask Mr. Steppuhn whether, from his recent experience, he feels that the need for the orifice vibrator results from the difficulty of obtaining a perfect solution of the Rhodamine WT ?

Steppuhn: I assume that the dye solution we received from the manufacturer is not a complete solution. In earlier studies using rhodamine dyes this was one of the major problems. The rhodamine molecule is an organic molecule and therefore it does not easily dissolve. A small rotating paddle within the orifice chamber would certainly help but still needs to be tried out.

Chemical method of water flow measurement in open channels

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Abstract. To extend the scope of discharge measurements, the Polish Hydrological Service has developed a chemical method of measurement. The method is characterized by the fact that, with regular dosing of a tracer ($Na_2Cr_2O_7 . 2H_2O$) the magnitude of constant concentration and the whole wave surface of the tracer is measured. In this way, two largely independent results are obtained. Particular attention was paid to adapting the method for field work while ensuring reliability of the final result.

METHODES CHIMIQUES DE MESURE DES DEBITS EN CANAUX DECOUVERTS

Résumé. Dans le but d'élargir l'éventail d'utilisation des mesures du volume d'écoulement, on a élaboré dans le service hydrologique polonais une méthode chimique des mesures. Cette méthode consiste à mesurer – pendant le dosage continu et invariable de l'indicateur ($Na_2Cr_2O_7 . 2 H_2O$) – la concentration fixe de cet indicateur, ainsi que toute la surface de l'onde de celui-ci ; on peut obtenir ainsi les deux résultats bien indépendants.

On a veillé surtout à ce que l'emploi de cette méthode, dans les travaux effectués en plein air, soit le moins incommode possible, assurant en même temps des conditions de travail convenables qui garantissent l'exactitude du résultat final.

MÉTODO QUÍMICO DE MIDICIÓN DE CAUDAL

Resumen. A fin de extender el campo de aplicación de las mediciones de volumen de caudal en el servicio hidrológico polaco, se trabajo en primer lugar con el método químico de medición. El método elaborado se caracteriza por el hecho de que con una dosificación regular de trazador $Na_2 Cr_2 O, 2H_2O$, se mide la magnitud de la presión constante y la superficie de onda total del trazador. De esta forma se obtienen dos resultados, en gran medida independientes. Se prestó particular atención al problema de la utilización del método para los trabajos en campo que presenta óptimas condiciones en cuanto a la fiabilidad del resultado final.

ХИМИЧЕСКИЙ МЕТОД ИЗМЕРЕНИЯ РАСХОДА ВОДЫ В ОТКРЫТЫХ РУСЛАХ

Резюме. Для расширения сферы измерений расхода польская гидрологическая служба разработала химический метод измерения. Этот метод характеризуется тем, что при правильном введении индикатора ($Na_2Cr_2O_7 . 2 H_2O$) измеряется величина постоянной концентрации и всей поверхности волны индикатора. Таким путем достигаются два в значительной степени независимых результата. Особое внимание было уделено применению этого метода для полевых работ, т. к. он обеспечивает надежность конечных результатов.

One of the most popular methods of measurement in open channels depends on determination of the active cross-section and velocities at various points of the cross-section. This "current meter method" is convenient since it can be used to measure both small and very large discharges; it is also economical, since it involves the use of simple, light devices. Despite these advantages, the method is not universal. Its particular imperfections appear in measurements of flow in uncontrolled rivers and mountain streams, in which the roughness of the bottom profile, by comparison with the small depth of water in the channel, particularly during low stages, leads to significant turbulence and does not permit precise measurement.

These difficulties may be overcome by the use of dilution gauging techniques. Numerous descriptions of such measurements can be found in the literature. The descriptions may be divided into various groups of methods, depending on the tracer used, on the method of injection, and on the nature of the subsequent analysis. Such descriptions, however, are chiefly adapted to the needs of universities and are not useful for the propagation of the method.

In general, such measurements are laborious and complex; there are additional difficulties due to the introduction of a range of devices necessary for measurement. Of the various possible procedures, the least complex should be selected. There would be a considerable advantage if such a procedure could be given a wide application to the field work of the National Institute for Hydrology and Meteorology, provided that it is capable of producing results within acceptable limits of error.

Attention was therefore given to the adoption of facilites, instruments and procedures to expedite the preparation of constant rate injections, the taking of standard samples, and the computing of the final result. The procedure which was evolved and the devices used in it are adapted to the means at the disposal of the Institute, with regard to instrument manufacturing, transportation and the purchase of the necessary chemicals and other accessories.

The procedure consists in the simultaneous application of two different methods, i.e. the method of constant rate injection and the gulp method which—for a slightly greater expenditure of work—provides two relatively independent measurements.

A characteristic feature of this method is that the tracer is introduced in a continuous and constant manner, with constant concentration (i.e the wave peak is flat), and the surface of this concentration wave is defined (see Fig. 1).

By recording the whole wave of concentrations it is possible to check whether the period of constant concentration, lasting from time T_Z to time $T_Z + T_S$, was established correctly.

The recording of concentrations—within the period of constant concentration—at three points of the cross-section permits one to observe the degree of spatial variations in concentrations and almost halves the error arising from this cause. The salt selected is sodium dichromate ($Na_2Cr_2O_7 \cdot 2H_2O$). The wave of concentrations is composed of three main phases: the phase of increasing concentration, the phase of constant concentration, and the phase of decreasing concentration (see Fig. 2).

According to the definition adopted tentatively by the Institute, the concentration n is the ratio of the solution volume to the total volume of solution and stream water. In conformity with the above definition, the concentration of the mother solution equals unity:

$$n_d = \frac{V}{V + O} = 1 .$$

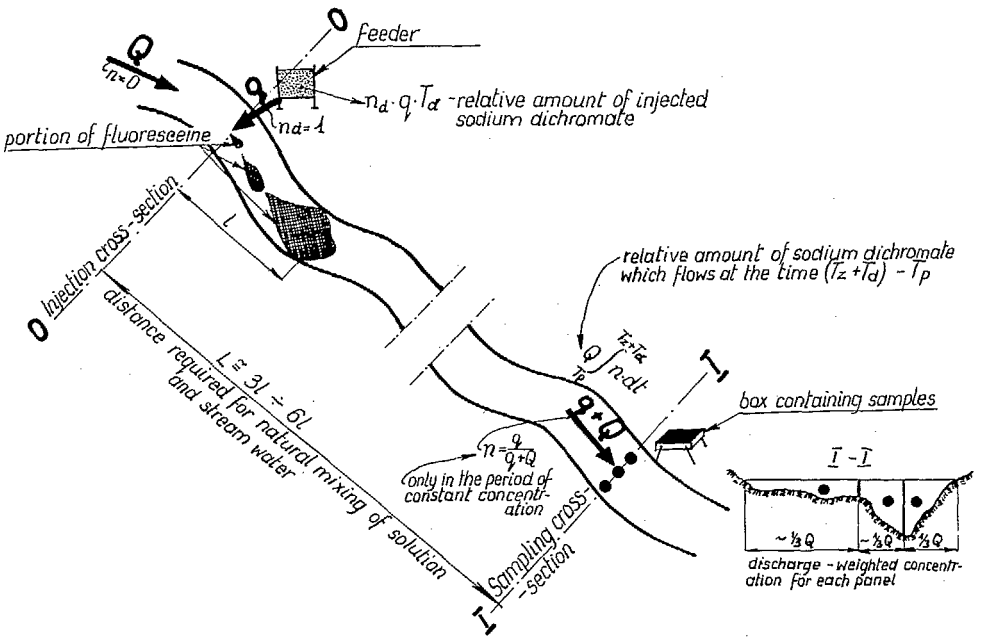


Figure 1. *General diagram of the method of measurement*

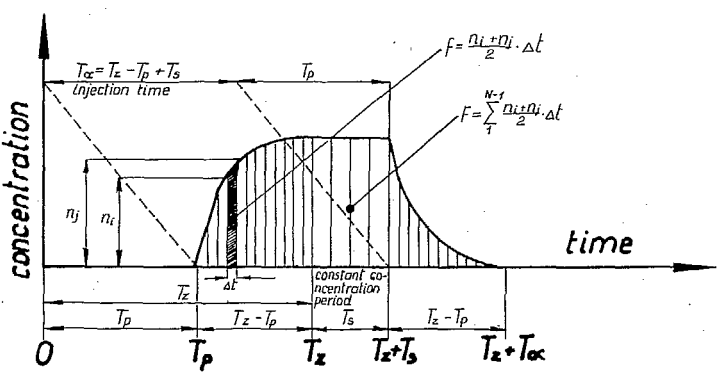


Figure 2. *The wave of concentrations of sodium dichromate*

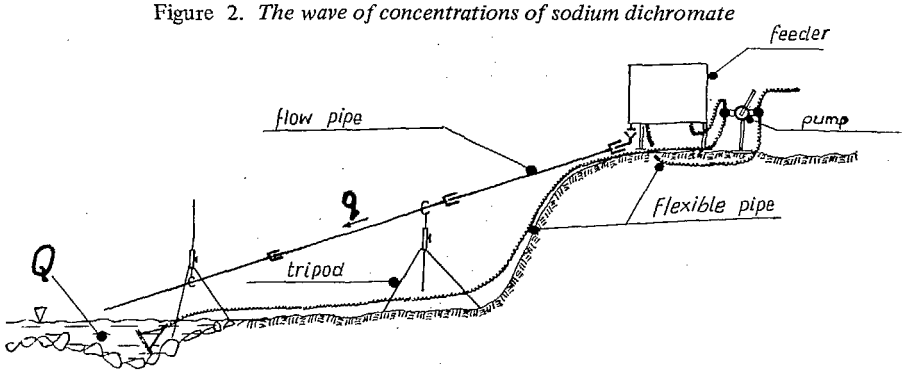


Figure 3. *The set-up for a constant injection rate*

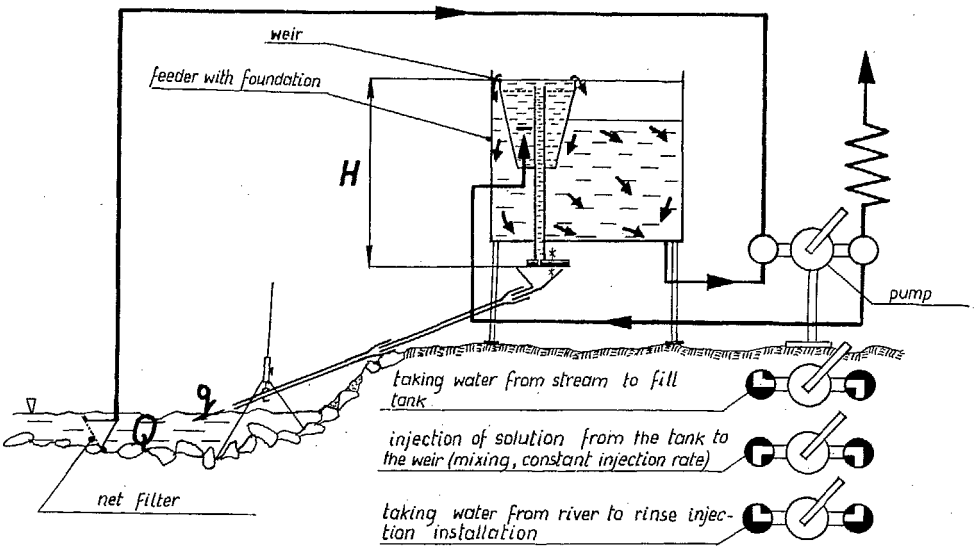


Figure 4. *Details of feeder operation*

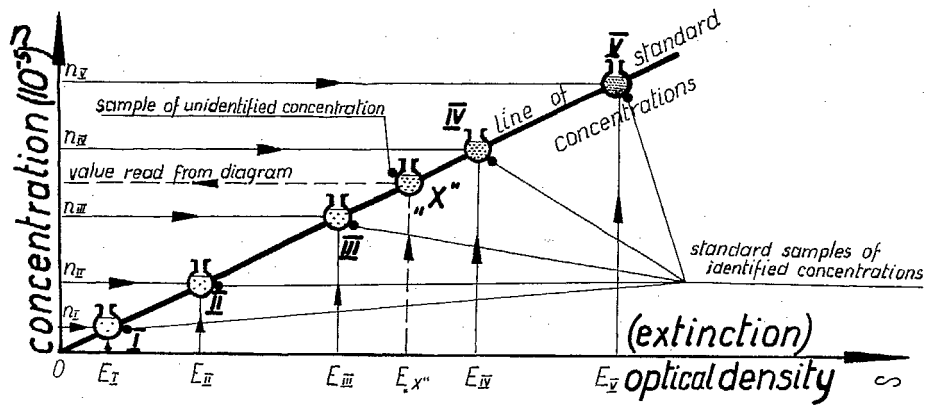


Figure 5. *Method for the measurement of sample concentrations*

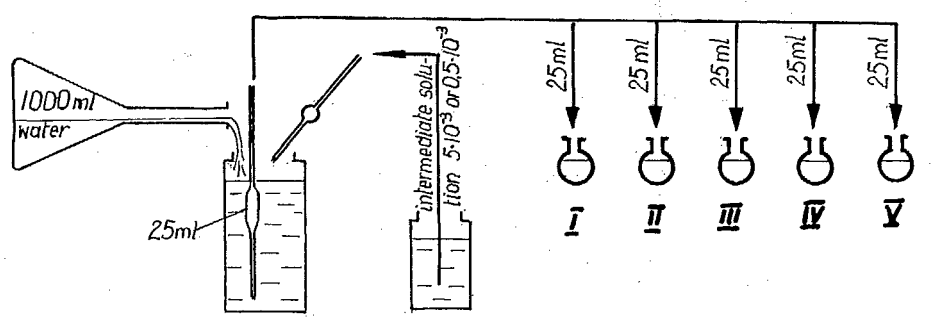


Figure 6. *The preparation of standard samples*

The concentration at the sampling cross-section in the period of constant concentration will be:

$$n = \frac{q}{q+Q}\ ;\ \text{from which}\ Q = \frac{q(1-n)}{n}.$$

Since n is very small, $(1-n) = 1$; consequently,

$$Q = \frac{q}{n} \tag{1}$$

where: Q = volume of water flow in stream (m^3/sec); q = outflow from feeder nozzle x 10^{-5} (m^3/sec); n = concentration of diluted solution x 10^{-5}.

In order to obtain a reliable concentration value, the arithmetical mean of 15 samples taken at three points of the cross-section and at five different times is adopted.

To estimate flow by means of the second method, the following data should be established: injected solution volume qT_α and the surface of the concentration wave F.

By comparing the relative amount of sodium dichromate in the solution injected into the stream with the relative amount of this tracer which will flow through the sampling cross-section, the following equation is obtained:

$$n_\alpha \cdot q \cdot T_\alpha = \int_{T_P}^{T_Z+T_\alpha} Q \cdot n \ \mathrm{d}t.$$

$n_\alpha = 1$ and Q is constant during the measurement time, so:

$$q \cdot T_\alpha = Q \int_{T_P}^{T_Z+T_\alpha} n \ \mathrm{d}t.$$

The value of the concentration integral $\int_{T_P}^{T_Z+T_\alpha} n \ \mathrm{d}t$ is equal to the concentration wave F, and is computed as a sum of trapezoid plots:

$$F = \frac{n_1+n_2}{2} \cdot \Delta t_1 + \frac{n_2+n_3}{2} \cdot \Delta t_2 + \ldots + \frac{n_{n-1}+n_n}{2} \cdot \Delta t_{n-1} = \sum_1^{n-1} \frac{n_i+n_j}{2} \Delta t$$

and finally:

$$Q = \frac{q \cdot T_\alpha}{\sum_1^{n-1} \frac{n_i+n_j}{2} \Delta t} \tag{2}$$

where: Q = volume of water flow in stream (m^3/sec); q = outflow from the nozzle x 10^{-5} (m^3/sec); T_α = constant rate injection time (min); Δt = time interval between successive samples (min); $n_i\, n_j$ = concentrations of successive samples (10^{-5}).

On comparing formulae 1 and 2 it is found that the same component q occurs in both relations. To be sure that the delivery of the nozzle did not change during the time of measurement, the outflow q should be checked; it will thus be possible to determine any significant error. To establish periods T_p and T_Z and the length required for the mixing section before taking the measurement, preliminary tests are made with fluoresceine ($C_{26}H_{10}O_5$), an intensive dye. Time T_p is the time needed for the tracer wave to travel from cross-section 0-0 to cross-section I-I, calculated from the moment of its injection into the stream; this is termed the "coloration appearance time". Time T_Z is the time counted from the moment of tracer injection into the stream at cross-section 0-0 to the moment at which the whole dye flows through cross-section I-I; this time may be called the "coloration time". Time T_s is a function of the time of the dye wave's passage ($T_Z - T_p$) through the cross-section I-I. For a péried $T_Z - T_p$ varying within the range of 6 to 40 min, time T_s changes from 4 to 20 min.

Sodium dichromate solution is injected into the river by means of a specially designed feeder (see Fig. 3). A feeder with a capacity of 90 litres is equipped with a system of nozzles which can be regulated in eight stages from about 0.18×10^{-5} m^3/sec to 5.4×10^{-5} m^3/sec. The use of the method in question for measuring flow from $Q = 0{,}02$ m^3/sec. to $Q = 4$ m^3/sec requires the use of a nozzle giving an outflow ranging from 0.7×10^{-5} m^3/sec to 1.9×10^{-5} m^3/sec in order to obtain plateau concentrations in the river from $n = 35 \cdot 10^{-5}$ to $n = 0.475 \times 10^{-5}$ ($n = q/Q$). The above limits of application of the chemical measurement method are not rigid, but difficulty is often caused when they are exceeded.

In view of the transportation problem, the feeder has been made light enough (28 kg) to enable two people to carry it safely from the car to cross-section 0-0. Apart from the injection of tracer, the feeder enables the performance of supplementary activities such as: taking water from the stream, mixing the solution, and washing and cleaning the injection equipment (see Fig. 4).

After acidifying the samples of unidentified concentrations, an alcohol reagent is added which causes a colour change. Depending on the amount of chromium contained in the sample, the colour of the sample is more or less reddish-violet. The amount of sodium dichromate, the volume of the prepared solution, the diameter of nozzle, and the formulation of standard samples are obtained separately from a special feeder table. Values of $T_\alpha = (T_z - T_p) + T_s$ and Q may be found from tables. The tables were prepared in such a way that, irrespective of the concentration of the mother solution and the nozzle diameter, maximum normal concentration of sodium dichromate in water will always amount to about 1 mg/litre in the period of constant concentration, which corresponds to a detectable reddish-violet sample colour.

Measurement of the sample concentrations depends on comparison of the sample colours with the colour of standard samples of known concentrations, shown in a schematic way in Fig. 5.

Measurement of the colour intensity is carried out by means of the Type KF-5 photocolorimeter manufactured in Poland. The measure of colour intensity is extinction, in other words optical density or absorption.

Many difficulties are encountered in preparing standard concentrations. A modification introduced by the Institute is as follows: for each measurement, the range of standard samples from the table is established. Samples are prepared mechanically from an intermediate solution of a concentration amounting to 5×10^{-3} or 0.5×10^{-3}. The intermediate solution 5×10^{-3} is prepared by injection of 5 ml of mother solution into 995 ml of water:





Figure 7 a, b. *Measurement in mountain rivers ; injection in cross-section*

$$n = \frac{5}{5 + 995} = 5 \times 10^{-3}.$$

For example, "D" samples are prepared successively by injecting into 1,000 ml of "pure" stream water 5, 10, 10, 10, 10 ml of intermediate solution of concentration amounting to 5×10^{-3}. After each injection of intermediate solution and mixing with water, a sample of 25 ml is taken. Samples taken this way will have the following concentrations: I–2.49; II–7.51; III–12.6; IV–17.6; V–22.7 (10^{-5}).The method of preparing standard samples is presented schematically in Fig. 6.

Stream water, to a greater or lesser degree, contains compounds which reduce chromium in the samples. In general, the process in question is slow but it may be the cause of additional error. For this reason, the photocolorimetric analysis of samples should be carried out as quickly as possible.

In practice, a group of three experienced scientific workers can perform two measurements of discharge in an eight-hour day, including travel time by car up to 100 km (assuming that a convenient site close to the gauging stations in known).

The above method can be used to measure discharges of up to a few cubic metres per second, on the assumption that:

– the water in the river is satisfactorily pure as far as its colour is concerned;
– the reach of natural mixing is not too long, and it is situated between successive tributary streams;
– the volume of discharge during measurement is constant.

Although numerous remedial measures exist which could allow a measurement in the absence of the above conditions, it is not to be recommended. The final result of the measurement is the mean value of the two methods. The divergency in results and the dispersion of concentrations of samples taken in the period of constant concentration permit one to make some estimation of the accuracy of the measurement. One may say that if the value of greatest deviation of concentration from the mean value does not exceed ± 10 per cent, one may expect that the general error of measurement will not exceed ± 5 per cent, and such an accuracy may be considered satisfactory.

Stream gauging with portable equipment

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Abstract. Methods are described for the collection of short-period discharge records from steep streams with flows up to 100 m^3s^{-1}.

Stage recorder installations: Excavation of a deep stilling well can often be avoided by locating the well some distance downstream from the measuring section. Connecting the well and stream through an inverted syphon also avoids trenching. The water level in wells with long, thin connexions to the stream is linearly damped and can be converted to true stream level.

Discharge measurements: Accurate discharges can be obtained by slug-injecting fluorescent tracers from volumetric pipettes or from prepared vials directly into the stream. At a downstream location where lateral mixing is assured, 10-20 samples are collected during the passage of the tracer in order to determine the time-concentration curve, from which discharge can be computed. This method is particularly suitable for spot measurements during floods.

STATION DE JAUGEAGE AVEC EQUIPEMENT PORTATIF

Résumé. Cette note décrit des installations et méthodes hydrométriques destinées à l'enregistrement des débits sur de courtes périodes présentant des pointes et qui s'appliquent à des débits allant jusqu'à 100 m^3/s.

Installation du limnigraphe. Une excavation profonde peut être évitée en plaçant le puits à une certaine distance en aval de la section de mesure. La connexion entre le puits et le cours d'eau peut être effectuée au moyen d'un long syphon inverse, ce qui évitera de creuser une tranchée.

Mesure du débit. Des débits exacts peuvent être obtenus par l'injection d'un traceur fluorescent directement à partir de pipettes volumétriques ou de fioles préparées à l'avance. A l'aval du point d'injection, lorsqu'un bon mélange latéral est assuré, 10 à 20 échantillons sont prélevés lors du passage du traceur. Cela permet de déterminer la fonction temps-concentration à partir de laquelle on calcule le débit. Cette méthode est parfaitement applicable pour des mesures instantanées pendant les crues.

AFORO DE CORRIENTES CON EQUIPO PORTÁTIL

Resumen. Se describen métodos para la medida de caudales en periodos cortos, en cursos de agua torrenciales con caudales hasta 100 m^3/seg.

Instalaciones de registro de nivel: Se puede evitar frecuentemente la instalación de un pozo de remanso profundo situando el pozo a alguna distancia aguas abajo de la sección de medición. La conexión del pozo y del curso de agua por medio de un sifón invertido evita también la excavación de zanjas. El nivel de agua en pozos con conexiones largas y estrechas al curso de agua está amortiguada linealmente y se puede convertir en nivel verdadero de la corriente.

Mediciones de caudal: Se pueden obtener caudales con precisión por medio de la inyección de trazadores fluorescentes desde pipetas volumétricas o a partir de viales directamente en la corriente. En un emplazamiento situado aguas abajo en el cual esté asegurada la mezcla lateral, se recogen 10 a 20 muestras durante el paso del trazador a fin de determinar la curva tiempo concentración, a partir de la cual se puede averiguar el caudal. Este método es particularmente adecuado para mediciones puntuales en crecidas.

ИЗМЕРЕНИЕ ПОТОКОВ С ПОМОЩЬЮ ПОРТАТИВНОГО ОБОРУДОВАНИЯ

Резюме. Описываются способы измерения (с записью данных) кратковременных расходов рек с большими расходами, до 100 м³/сек. Установки самописцев уровня, устройства глубоких успокоительных колодцев часто можно избежать путем размещения колодца на некотором расстоянии вниз по течению от измерительного створа. Связывая уровни воды в колодце и в реке с помощью перевернутого (обратного) сифона, можно избежать также прорытия траншеи.

Уровень воды в колодцах при наличии длинных и тонких соединительных труб с рекой линейно гасится и может быть обращен в истинный уровень ве реке.

Измерение расхода: точные расходы могут быть получены мгновенным (единовременным) вводом флюоресцентных красителей из измеряющих объем пипеток или из специальных сосудов, непосредственно в поток. В измерительном створе, где достигнуто достаточно полное перемешивание, берется от 10 до 20 проб, чтобы определить кривую зависимости концентрации от времени, по которой вычисляется расход. Этот метод особенно пригоден для измерений во время паводков.

INTRODUCTION

It is not generally appreciated that under certain circumstances some of the modern dilution gauging methods permit the execution of accurate discharge measurements with very little field equipment. Sometimes it is also possible to install stage recorders in such a way that most of the rather expensive construction and excavation work normally associated with permanent recorder installations is avoided. The writers have experimented with inexpensive, portable stream gauging equipment over the last six years. A detailed account of the experiences together with a survey of the literature on this subject is contained in Church and Kellerhals (1970). Two somewhat unconventional methods which were found to be very effective for stage measurements and discharge measurements respectively, are described here.

The portable stage-recorder installation is suitable for the collection of discharge records during expeditions into remote areas or it may be used to advantage at short-term stream gauging sites in hydrometric networks combining permanent bench-mark stations with roving short-term stations. The system is not limited to any particular type or size of stream, although it is most effective on steep mountain streams.

The portable dilution-gauging method is somewhat more restricted in application. It is most suitable for highly turbulent streams (culverts or drop-structures in flat streams, steep mountain streams) with discharges less than approximately 100 $m^3 s^{-1}$. The streams should also not carry high concentrations of clay minerals. The method is recommended for the collection of discharge data during rare flood events in small streams which are not normally being gauged, or for the establishment of stage-discharge rating curves at remote or inaccessible sites.

STAGE MEASUREMENTS

The proposed system is shown schematically in Fig. 1, while Fig. 2 shows a field installation. The main feature is the connexion between the stream and the stilling

well which consists of two steadily rising hoses leading to an air reservoir at the highest point, where the steady, small flow of air which comes out of solution in the stream water can accumulate. This inverted syphon keeps the installation operational between periodic removals of air from the reservoir. Any float-operated water level recorder may be used. If the air reservoir is fabricated from a fairly soft, thick-walled plastic, the installation will withstand night frosts without damage but it is not easily winterized. The writers have successfully used several of these installations on steep mountain streams. The gauge wells were placed outside the channel only a short distance downstream from the inlet. Only minor excavation of 1 or 2 feet (30 or 60 cm) was required. Even though these streams were very turbulent and irregular in cross-section, stable stage-discharge relations could generally be established at carefully selected pools which were controlled by bedrock or by very large boulders.

The long, relatively thin hose connexion between the stilling well and the stream eliminates turbulent level fluctuations from the recorder trace.

On larger and flatter streams, where the excavation of a stilling well is unavoidable, the inverted syphon technique still eliminates the trenching which is usually required for a low-level pipe connexion to the stream. Recorder stilling wells on rivers with very permeable banks can often rely on seepage to assure adequate hydrostatic connexion with the stream.

If rapidly fluctuating water levels are to be recorded on wells using inverted syphons or seepage for the connexion with the stream, it may be necessary to eliminate the damping effect of the long hoses from the records. Under normal circumstances the flow in the hose is laminar and the inertia of the flowing water is negligible. The response is then governed by the following linear differential equation

$$h = -b\frac{\mathrm{d}h}{\mathrm{d}t} \tag{1}$$

in which h is the elevation difference between the stream and the stilling well (Fig. 3), b is a constant, and t is time. The solution for a constant stream level is

$$h = h_0 \exp[(t_0 - t)/b]. \tag{2}$$

The response of a recorder installation is tested by dumping some water into the well. Starting at an arbitrary time t_0, one observes the decline of the stilling well level. A plot of $(t\text{-}t_0)$ versus h/h_0 should give a straight line on semi-log paper, but some deviation is possible since only laminar pipe friction losses obey the above equation and there are certain other losses present. The slope of the line is equal to $(-b)$. The stage records can be corrected to true stream level by adding $b(\mathrm{d}h/\mathrm{d}t)$ to the recorder trace, $\mathrm{d}h/\mathrm{d}t$ being the slope of the recorder trace. Fig. 4 shows some typical gauge response curves. Instead of plotting a curve one can measure the time Δt it takes the stilling well to drop from an arbitrary h_1 to an arbitrary h_2 and compute b, as follows

$$b = \frac{\Delta t}{\ln(h_1/h_2)}. \tag{3}$$

A simple manual gauge constructed from a Perspex tube (Fig. 4) and set in or near the edge of the creek serves as a check for the automatic recorder. The thin hose connexion between the stream and the tube need not contain an air reservoir. It is,

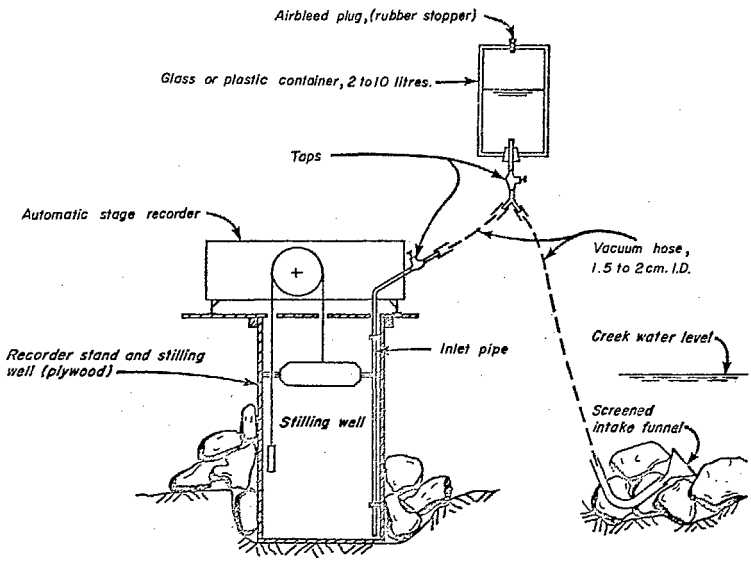


Figure 1. *Schematic view of stage recorder installation*



Figure 2. *Stage recorder with inverted siphon*

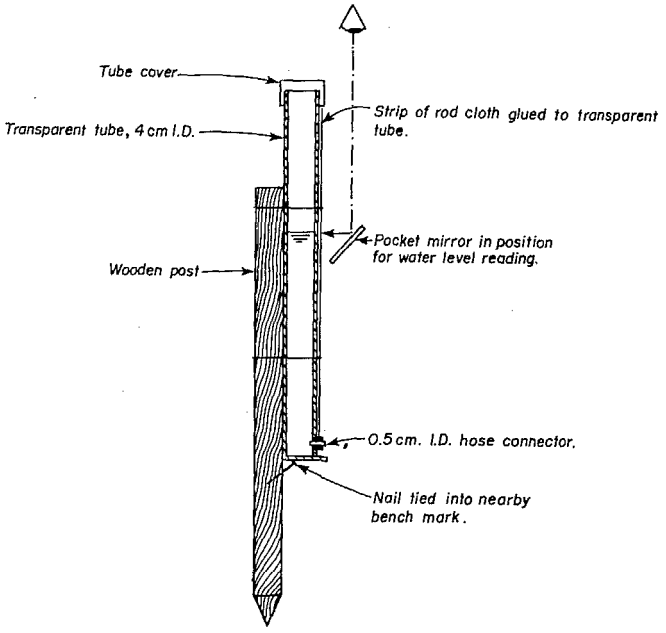


Figure 3. *Gauge response curves*

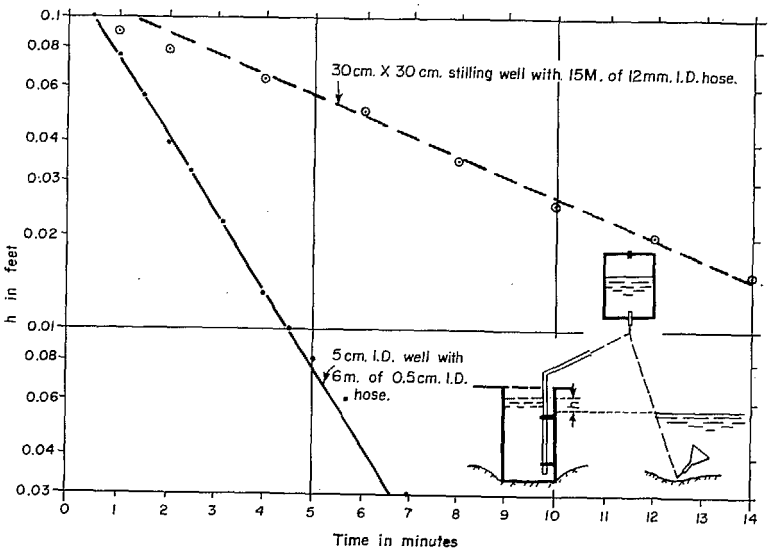


Figure 4. *Manual tube gauge*

however, important to purge air from the hose before each reading by dumping some water into the gauge tube. Care should also be taken to read the water level in the centre of the tube and not some surface-tension-dependent line on the edge of the tube.

DISCHARGE MEASUREMENTS

Tracer methods are generally most suitable for the gauging of highly turbulent and irregular streams. In particular, the method based on slug injection of a fluorescent tracer provides great advantages under adverse field conditions. The simple field procedure and very light field equipment (Fig. 5) can be reliably used in harsh weather conditions which often are present during important flood events, and the method is not impaired by restriction of access to only one bank of the stream.

The principle of the method is explained in Replogle *et al.*, 1966. A slug of dye of mass M is injected into a stream of steady, but unknown discharge, Q. At some point downstream, where complete lateral mixing has been achieved, one observes the time-concentration curve, $C(t)$, of the passing dye cloud. The discharge is computed from

$$Q = \frac{M}{\int_{t_s}^{t_e} C \, dt} \tag{4}$$

in which t_s and t_e are the starting points and end points of the observed dye cloud (Fig. 6).

Tracer selection. Several suitable dyes are readily available. Wilson, 1968 a, gives a list of the more common ones, showing their strong and weak points. He also discusses and references the health hazards associated with tracer dyes and gives certain values for maximum tolerable concentrations.

The writers have consistently used the Rhodamine WT, 20 per cent solution[1], and found it easy to handle and resistant to adsorption by stream sediments. Photochemical decay in the stream and in stream samples is negligible. The concentrated dye can be stored for prolonged periods without noticeable loss of strength. Concentrations as low as 10^{-10} kg/litre can be analysed to an accuracy of two significant figures.

Tracer injection. The amount of tracer needed for most discharge measurements is low, values between 10 cm^3 and 500 cm^3 are common. Wilson (1968b) gives an empirical formula which is, after conversion to metric units

$$M = 2.25 \cdot 10^{-6} \cdot \frac{Ql}{v} C_p \tag{5}$$

where M is the required amount of Rhodamine WT, 20 per cent solution, Q is the estimated discharge in m^3s^{-1}, l is the length of the reach between injection point and sampling point, and v is the estimated mean stream velocity in ms^{-1}. C_p is the desired peak concentration in units of 10^{-9} kg/litre.

1. Produced by the DuPont de Nemours Co.

Equation 5 is largely based on travel time measurements in intermediate to large rivers. The writers' data obtained from steep mountain streams exclusively support it closely (see Fig. 7). C_p values around 3 to 6 are recommended in populated areas. In remote areas values of 20 to 50 may be advisable as the dye is then visible. Periodic short exposures to such high dye concentrations appear to be harmless to stream biota.

It was found practical to carry the dye to the field in a bottle. After selection of a proper amount M, the dye can be injected from volumetric pipettes directly into the stream, taking due account of the specific gravity of Rhodamine WT, 20 per cent solution, which is 1.11. Alternatively, one can carry several vials, containing a range of accurately weighed amounts of dye.

Sampling. The most difficult aspect of stream gauging by dilution techniques is the selection of an adequate distance between the injection and sampling points. Too short a reach results in erroneous computed discharges due to incomplete mixing, while too long a reach wastes dye and prolongs the measurement. For highly irregular streams there are no rules to guide the observer and judgement has to be developed through practice. Several empirical formulas have been developed for streams with reasonably uniform flow. They are listed and compared in Church and Kellerhals, 1970.

The sampling is done by collecting 10 to 20 samples of 50 cm^3 to 100 cm^3 each during the passage of the dye cloud. If the dye is visible, fewer samples are required because the time lag to the arrival of the peak concentration, t_p, can be guessed reasonably accurately in the field. The writers' data suggest that sampling should be extended to approximately 2.5 times t_p (see Fig. 8). This assures in most cases that the concentration will have dropped to below 5 per cent of C_p. By taking a few simultaneous samples at various locations in the cross-section one can check whether adequate lateral mixing has been achieved. Initially, sampling at several successive points downstream during the same measurement helps the observer to develop reliable judgement.

Analysis. In the laboratory the stream samples are analysed for dye concentration on a fluorometer. The produce is extremely simple, several excellent outlines are available (G.K. Turner, 1968; Wilson, 1968a).

Finally, the time-concentration values have to be integrated for the evaluation of discharge according to equation 4. This can be done by plotting the curve and measuring the area under it or by standard numerical techniques.

Several authors (Florkowski *et al.*, 1969; Kellerhals, 1970) have found that the final decline of a time-concentration curve is approximately negative-exponential. By fitting such a curve through the last few observed points, the integral of equation 4 can be extended to infinity.

A Fortran IV computer programme to perform the discharge computations is available on request from the senior author. It integrates the time-concentration data according to a 3rd-order method including an extension of the decline to infinite time with a negative exponential curve fitted to the last four observed points. The programme also computes the arithmetic mean and harmonic mean of the time-concentration curve, which correspond to the mean residence time and to the mean travel time of the dye cloud respectively (Thackston *et al.*, 1967). The mean velocity of the dye cloud is evaluated by dividing the length of the test reach by the mean travel time.

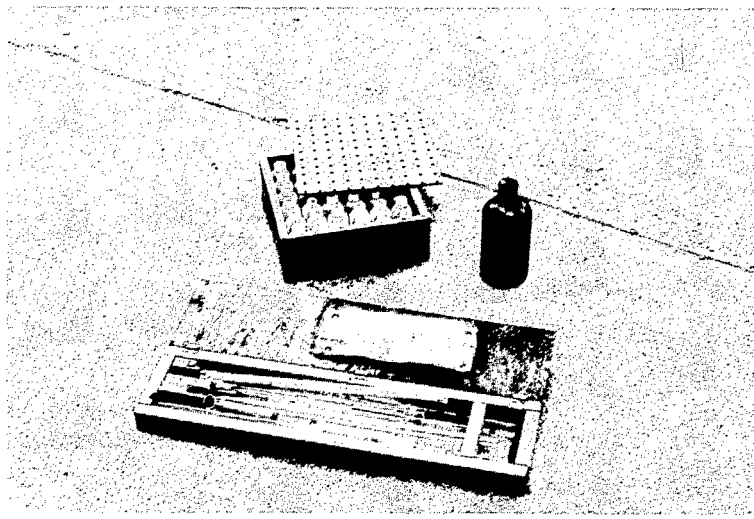


Figure 5. *Equipment for Rhodamine WT shug injection tests*

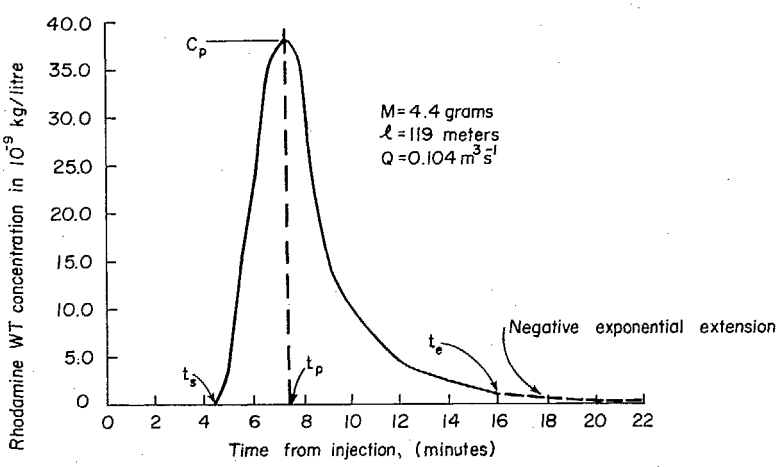


Figure 6. *Typical time-concentration curve*

Limitations. Several factors tend to limit the applicability of this dilution-gauging method to relatively small and clear streams.

The time and mixing length requirements for a measurement increase rapidly with stream size and tend to become prohibitive at discharges in the order of 100 $m^3 s^{-1}$. In steep, highly turbulent streams this limit may be somewhat higher and in slow-flowing streams with a high width-depth ratio it may be much lower.

Implicit in equation 4 is the assumption that all the injected dye passes the sampling location. If the travel time of the dye is large or if the stream carries a high suspended load (particularly clay minerals), dye losses may become appreciable.

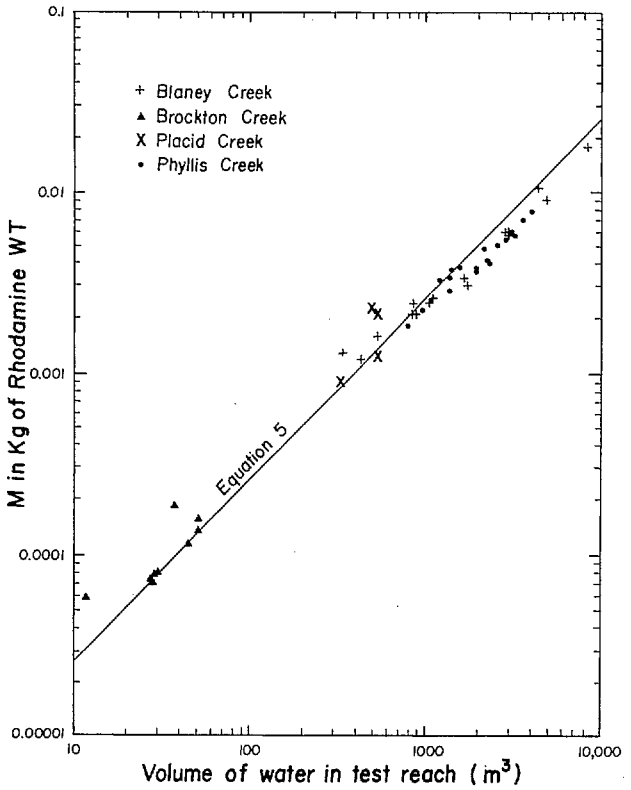


Figure 7. *Empirical relation between volume of water in stream and amount of dye*

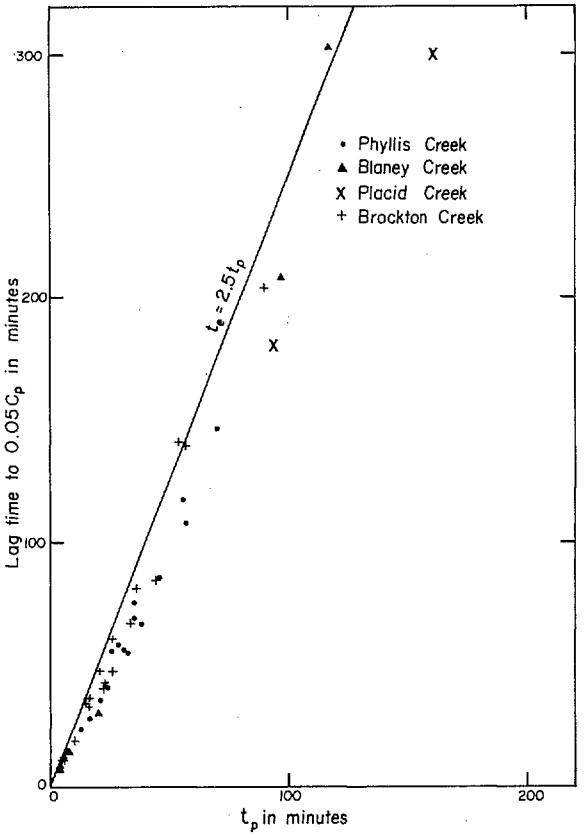


Figure 8. *Empirical relation between t_p and t_e*

BIBLIOGRAPHY / BIBLIOGRAPHIE

Church Michael; Kellerhals, Rolf 1970. Stream gauging in isolated areas using portable equipment. *Technical Bulletin No.25,* Dept. of Energy, Mines and Resources, Inland Waters Branch, Ottawa. (Available on request.)

Florkowski, T. *et al.,* 1969. The measurement of high discharges in turbulent rivers using tritium tracer. *Journal of Hydrology,* vol. 8, p. 249-64.

Kellerhals, R. 1970. *Runoff concentration in steep channel networks.* Ph.D. thesis, University of British Columbia, Vancouver 8, B.C.

Replogle, J. A., Myers, L. E.; Brust, K. J., 1966. Flow measurement with fluorescent tracers. *American Society of Civil Engineers. Proceedings. Journal of the Hydraulics Division,* vol. 92, no. HY5, p. 1-16.

Thackston, E. L., *et al.,* 1967. Least squares estimation of mixing coefficients. *Journal of the Sanitary Engineering Division,* Am. Soc. of Civil Engineers, vol. 93, no. SA3.

Turner, G. K. Associates, 1968. Fluorometry in studies of pollution and movement of fluids. *Traces,* Acc. no. 9941.

Wilson, J. F., Jr., 1968a. Fluorometric procedures for dye tracing. United States Geological Survey. *Techniques of Water Resources Investigations.* Book 3. *Applications of Hydraulics,* Chapter A12.

Wilson, J. F., Jr., 1968b. An empirical formula for determining the amount of dye needed for time-of-travel measurements. U.S. Geological Survey, *Professional Paper* 600-D, p. D54-D56.

Precision and bias of the results of dilution gaugings

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Abstract. The effects of a number of sources of error on the precision and bias of results from dilution gaugings are examined theoretically for the constant-rate-injection technique. From these considerations, recommendations are made for the precautions that should be observed to minimize the errors of the estimated discharges.

PRECISION ET ERREUR SYSTEMATIQUE DES RESULTATS DE JAUGEAGES PAR LA METHODE DE DILUTION

Résumé. Cette note examine de façon théorique les effets d'un certain nombre de sources d'erreur sur la précision et l'erreur systématique des jaugeages par la méthode de dilution, injection à débit constant.

De ces considérations on tire des recommandantions sur les précautions à prendre pour réduire les erreurs des débits estimés.

PRECISIÓN Y DISPERSIÓN DE LOS RESULTADOS EN LOS AFOROS POR DILUCIÓN

Resumen. Se examinan teóricamente los efectos de cierto número de fuentes de error en la precisión y dispersión de los resultados de aforos por dilución, correspondientes al método de inyección constante. A partir de dichas consideraciones, se realizan recomendaciones para las precauciones que se deberían observar a fin de reducir al mínimo los errores en los caudales estimados.

ТОЧНОСТЬ И СИСТЕМАТИЧЕСКИЕ ОШИБКИ РЕЗУЛЬТАТОВ ИЗМЕРЕНИА РАСХОДОВ СПОСОБОМ СМЕШЕНИЯ

Резюме. Вляние многих источников ошибок на точность и систематические ошибки способа смешения исследуются теоретически для методики с вводом индикатора с постоянной интенсивмостью. На основании этого даются рекомендации, которые должны быть учтены, чтобы свести к минимуму ошибки измерений.

1. INTRODUCTION

The basis of the constant-rate-injection dilution gauging technique is simple. In essence, a solution containing a known concentration of a suitable chemical is injected into the stream whose discharge is to be measured, and the concentration of this chemical is then measured downstream of the injection site. Provided that the downstream site is chosen so that complete mixing of the injected solution and the stream has occurred, the discharge is easily calculated (see equation 1). There are, however, several sources of error that may affect the accuracy of gauging results. The

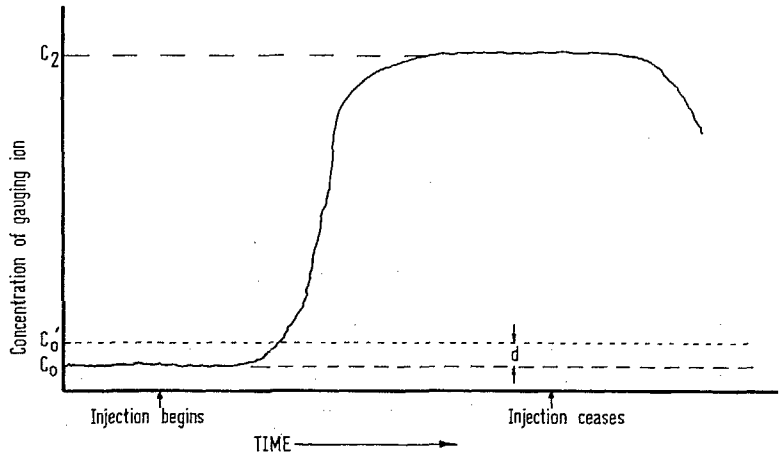


Figure 1. *Diagrammatic representation of constant-rate-injection dilution-gauging technique*

purpose of this paper is to consider the effects of such errors so that recommendations can be made on the precautions necessary to minimize the errors of the estimated discharges. Figure 1 shows a diagrammatic representation of the technique.

2. ERRORS OF RESULTS

2.1. General method for calculating the stream discharge

The basic equation for calculating the stream discharge is :

$$\tilde{Q} = \frac{10^6 \, q \, (C_1 - C_2)}{C_2 - C_0} \tag{1}$$

where $\tilde{Q}$ = estimated value of the stream discharge (m^3/s) ; q = rate of injection of the concentrated solution of the gauging ion (ml/s) ; C_1 = concentration of the gauging ion in the concentrated solution (mg/l) ; C_2 = steady-state concentration of the gauging ion in the stream at the downstream sampling site (mg/l) ; and C_0 = apparent concentration of the gauging ion that would be present in the stream at the downstream site if no injection had been made (mg/l).

In practice, C_1 is very much larger than C_2 and negligible error is introduced by neglecting C_2 in the numerator of equation 1. Inspection of this equation shows that C_1 must not be biased with respect to C_2. Now C_2 must be determined by chemical analysis of stream waters that may contain substances that interfere with the analysis. Therefore, the WRA and many others have attempted to eliminate the error caused by interfering substances by diluting the concentrated solution of the gauging ion with a sample of the stream water (background water) collected upstream of the injection point.

The diluted concentrate and downstream sample then have essentially the same composition and are analysed with reference to standards also prepared from the

same background water. It is impossible to determine C_0, and therefore an approximation is made by equating the concentration of the gauging ion, C_0, in the background water to C_0.

On this basis, equation 1 can be rewritten in terms of the measured variables:

$$\tilde{Q} = \frac{10^6\, qk\, (C_1' - C_0')}{C_2 - C_0'} \tag{2}$$

where k = the dilution factor = $C_1(C_1 - C_0')$; C_1' = concentration of the gauging ion in the diluted concentrate (mg/l), C_0' = apparent concentration of the gauging ion in the background water (mg/l).

The random and systematic errors of q and k are expected to be much smaller than those of the measured concentrations, and for simplicity it is assumed that the factor $10^6\, q\, k$ is essentially constant and equal to k. Of course, experimental care is required to ensure adequate control of q and k but there is no inherent problem in achieving this control. The equation of basic interest can therefore be written:

$$\tilde{Q} = K \cdot \frac{C_1' - C_0'}{C_2 - C_0'} \,. \tag{3}$$

Equation 3 may now be used to calculate the effects of errors in three concentrations on the error of $\tilde{Q}$. Systematic and random errors are considered separately in sections 2.2 and 2.3 respectively, and the practical implications of these considerations are discussed in section 3. Throughout the following treatment, it is assumed that the stream discharge is essentially constant during the period of the gauging.

2.2. Systematic errors (bias)

2.2.1. Analytical errors. Several types of error may affect the individual values found for C_1' and C_2 ; e.g. (a) adsorption of the gauging ion by the walls of the sample containers will give low results, (b) leaching of the gauging ion from the sample containers gives high results (this effect has been reported [ref. 1] for one of the more commonly used gauging ions, lithium), (c) substances present in the stream water may cause high and/or low results. The magnitude of such systematic errors will depend on the concentration of the gauging ion in the sample to be analysed. Thus if C_1' is closely similar to C_2, these errors should be essentially the same for both types of sample and will therefore have no important effect on the concentration ratio. For example, suppose that $C_1' = C_2$ and that both concentrations are subject to the same systematic error, e; then:

$$\frac{C_1'}{C_2} = \frac{C_1' - C_0'}{C_2 - C_0'} = \frac{(C_1' - e) - C_0'}{(C_2 - e) - C_0'}$$

i.e., e has no effect on $\tilde{Q}$ when $C_1' = C_2$.

In practice, the background water is used to set the zero reading of the analytical system used. Thus, providing the calibration curve for the analytical system is

rectilinear and constant, uncertainty in its slope is unimportant because the ratio $[(C_1' - C_0')/(C_2 - C_0')]$ is equal to the ratio of the analytical readings for C_1' and C_2 whatever the slope of the calibration. There is, therefore, no need to convert instrument readings to concentrations nor to establish the slope of the calibration curve. It is necessary only to ensure that this curve is rectilinear and passes through the origin ; even the latter condition is not necessary when $C_1' = C_2$. Constancy of the slope of the calibration curve is desirable but not essential (see section 3.1.1(b)).

2.2.2. Background errors. The concentrated solution of the gauging ion is diluted with background water. The experimental value for $(C_1' - C_0')$ will, therefore, be an unbiased estimate of the true value provided that the apparent concentration of the gauging ion in the portion of diluting background water is the same as that in the portion used for setting the zero reading of the analytical system. Any factors tending to cause differences between these two portions must therefore be eliminated or minimized.

The true increase in the concentration of the gauging ion at the downstream site is equal to $(C_2 - C_0)$; the experimental estimate for this increase is $(C_2 - C_0')$. Thus, if $C_0 \neq C_0'$, the latter estimate will be biased ; if $C_0 = C_0' - d$ it can be shown that the percentage bias, B_b, in the calculated value for $\bar{Q}$ is given by:

$$B_b = \frac{100d}{C_2 - C_0 - d} \% . \tag{4}$$

If, as seems reasonable, conditions are chosen so that d is considerably smaller than $(C_2 - C_0)$, equation 4 can be written approximately as:

$$B_b = \frac{100\,d}{C_2 - C_0'} \%. \tag{5}$$

Thus, if d is equivalent to 1 per cent of $(C_2 - C_0')$, the bias in $\bar{Q}$ will also be 1 per cent. It is clearly desirable to minimize d in any dilution gauging; this aspect is discussed further in Section 3.3.

2.2.3. Losses and/or gains of gauging ion in the stream. Equation 1 is obtained from a mass balance for the gauging ion assuming that its only sources are the stream itself upstream of the injection point and the concentrated solution that is injected. Thus, errors will arise if losses and/or gains of the gauging ion occur between the injection and downstream sites. Suppose that the net loss of the gauging ion is 1 per cent of the concentration that would have been obtained with no losses and gains. It can be shown that the percentage bias, B_1, in the calculated value of $\bar{Q}$ is given by:

$$B_1 = \frac{100\,L}{100 - L} \%. \tag{6}$$

Thus, a loss of 1 per cent of the gauging ion causes a bias in the calculated discharge of 1 per cent. It is clearly desirable for gauging, therefore, to use ions that are not subject to losses or gains. The former requirement is well recognized but the possibility of the latter must not be overlooked ; it could arise, for example, from the discharge of industrial effluents between the injection and downstream sites.

2.3. Random errors (precision)

The symbols $V(x)$ and $\sigma(x)$ are used in the following to denote the variance and standard deviation of single experimental measurements of the random variable x; the symbol $D(x, y)$ is used to denote the covariance of the random variables x and y. From equation 3 and the Law of Propagation of Errors [ref. 2] we can write:

$$V(\tilde{Q}) \approx \frac{K^2 V(C_1')}{(C_2 - C_0')} + \frac{K^2 V(C_2) \cdot (C_1' - C_0')^2}{(C_2 - C_0')^4} + \frac{K^2 V(C_0) \cdot (C_1' - C_2)^2}{(C_2 - C_0')^4}$$
$$+ \frac{2K^2 D(C_1', C_0') \cdot (C_1' - C_2)}{(C_2 - C_0')^3} - \frac{2K^2 D(C_2, C_0') \cdot (C_1' - C_0')(C_1' - C_2)}{(C_2 - C_0')^4}$$
$$- \frac{2K^3 D(C_1', C_2) \cdot (C_1' - C_0')}{(C_2 - C_0')^3} . \qquad (7)$$

It is now assumed that random fluctuations in the apparent concentration of the gauging ion just upstream of the injection site (at the point where the background water is collected) and at the downstream site are equal but uncorrelated. Under this assumption, it can be shown that:

$$D(C_1', C_0') = V_s$$
$$D(C_1', C_2) = 0 \qquad (8)$$
$$D(C_2, C_0') = 0$$

where V_s = variance of the apparent concentration of the gauging ion in the stream.

Substitution of equation 8 in 7, some re-arrangement, and taking the square root of both sides of the equation leads to:

$$\frac{\sigma(\tilde{Q})}{(\tilde{Q})} \cdot 100 \approx 100 \sqrt{\left[\frac{V(C_1')}{(C_1' - C_0')^2} + \frac{V(C_2)}{(C_2 - C_0')^2} + \right.}$$
$$\left. + \frac{V(C_0') \cdot (C_1' - C_2)^2}{(C_2 - C_0')^2 (C_1' - C_0')^2} + \frac{2V_s \cdot (C_1' - C_2)}{(C_2 - C_0')(C_1' - C_0')^2} \right] . \qquad (9)$$

Random fluctuations will occur in C_1', C_2 and C_0' for two reasons, i.e., the imprecision of the analytical system and variations in the apparent concentration of the gauging ion in the stream. The latter are attributable to natural changes in the stream quality for q and Q have been assumed to be constant. Let the suffices a and s denote the analytical and stream sources of random error. From previous assumptions we can write:

$$V_s(C_1') = V_s(C_2) = V_s(C_0') = V_s .$$

Further, if n analyses are made on each of m samples of each of the three types (i.e. C_0', C_1', C_2)

$$V(\bar{C}_1') = \frac{V_a(C_1')}{mn} + \frac{V_s}{m}$$

$$V(\bar{C}_2) = \frac{V_a(C_2)}{mn} + \frac{V_s}{m} \qquad (10)$$

$$V(\bar{C}_0') = \frac{V_a(C_0')}{mn} + \frac{V_s}{m}$$

where the bar over a symbol denotes the mean value of the variable.

Equation 10 may be substituted into 9, and chosen numerical values inserted to evaluate the right-hand side which is approximately equal to the coefficient of variation of $\bar{\bar{Q}}$. However, it has already been seen that it is desirable for C_2 to equal C_1', and this condition then allows a useful simplification when equation 10 is substituted into 9, i.e.,

$$\frac{100\sigma(\bar{\bar{Q}})}{\bar{\bar{Q}}} \approx \frac{1.41\,\sigma_a(C_2)}{(C_2 - C_0')\sqrt{m}} \sqrt{\left[\frac{1}{n} + \frac{\sigma_s^2}{\sigma_a^2(C_2)}\right]}. \qquad (11)$$

Using the symbol $R(x)$ to denote the coefficient of variation of the variable x, we have finally :

$$R(\bar{\bar{Q}}) \approx \frac{1.41\,R_a(C_2)}{\sqrt{m}} \frac{C_2}{C_2 - C_0'} \sqrt{\left[\frac{1}{n} + \frac{\sigma_s^2}{\sigma_a^2(C_2)}\right]}. \qquad (12)$$

Equation 12 allows theoretical consideration of the effect of the various relevant parameters on the precision of the result, $\bar{\bar{Q}}$, of a gauging. It is emphasized that the equation is based on the condition $C_1' = C_2$ but it represents a reasonable approximation for $0.8 \leqslant C_1'/C_2 \leqslant 1.2$.

3. PRACTICAL IMPLICATIONS

3.1. Eliminating or minimizing systematic errors

3.1.1. Dilution and analysis of the concentrated solution of the gauging ion. Section 2.2.1 showed the desirability of making $C_1' = C_2$, and of ensuring that the magnitude of any systematic errors affecting C_1' and C_2 be equal for both types of sample.

(a) Dilution. When the stream discharge is known to within 10 to 20 per cent, there should be little problem in choosing a suitable dilution of the concentrated solution of the gauging ion to ensure that $C_1 \approx C_2$. When no estimate of the discharge is available, it is useful to prepare several, say three, different dilutions so that the one closest to C_2 may be used for determining the discharge. These different dilutions may be used also for checking the linearity of the analytical calibration curve (see section 3.1.2.).

Section 2.2.2 showed that the apparent concentrations of the gauging ion should be equal in the two portions of background water used for diluting the concentrated solution and for setting the zero reading of the analytical system. There should be no

problem in ensuring this equality except when the background water contains appreciable amounts of suspended and/or collodial materials that cause an apparent content of the gauging ion. Under such conditions, great care should be taken to ensure either that such materials are removed (e.g., by filtration) before analysis or that the same concentration of undissolved materials enters both portions of the background water. For the latter approach, vigorous shaking of the background water container before rapid withdrawal of any portions should usually be adequate.

(b) Analysis. Three factors, in general, need to be considered though none or only some of them may be important depending on the gauging ion ; these factors are:

1. Interference effects by other constituents of the samples;
2. Loss or gain of the gauging ion in the samples between sampling and analysis (e.g. by adsorption on the walls of the sample containers);
3. Changes in the chemical form of the gauging ion between sampling and analysis (e.g., Cr (VI) → Cr(III)).

Factor (1) should be essentially eliminated by the methods of dilution of the concentrated solution and calibration of the analytical system. Only detailed investigations can show the importance of factors (2) and (3) for particular ions. Until such investigations have revealed the extent to which precautions may be relaxed, the diluted concentrate should be stored in the same type of container and for the same time before analysis as the downstream and background samples. Whenever possible, one should use analytical systems whose response to the gauging ion is independent of the chemical form of that ion. It follows from these considerations that it is preferable to use ions that are stable, that exist in only one chemical form, that are not adsorbed by the walls of the sample containers, and whose concentrations in samples are unlikely to be affected by contamination. It is emphasized, however, that these properties are not the only ones involved in selecting a gauging ion; practical considerations such as solubility, cost and concentration of the ion in the stream to be gauged are also important.

It is a sensible precaution to alternate the analysis of the diluted concentrate and the downstream samples so that the effect of any temporal changes in the slope of the calibration curve is minimized if not eliminated.

3.1.2. Calibration of the analytical system. If $C_1' = C_2$, the slope and intercept of a constant rectilinear calibration curve have no effect on the ratio $(C_1' - C_0') / (C_2' - C_0')$. It is, therefore, desirable to use analytical systems giving such calibration curves. Provided such systems are used, there is, in principle, no need to establish the calibration curve with any great precision. All that is necessary is direct measurement of the analytical readings, A, corresponding to C_1' and C_2 and measured relative to the response for C_0', i.e.,

$$\tilde{Q} = K \cdot A(C_1') / A(C_2).$$

In practice, regular checks that the calibration curve is remaining rectilinear are desirable but they need not be made with great precision so long as $C_1' \approx C_2$. The three different dilutions of the concentrated solution mentioned in Section 3.1.1(a) would allow an adequate check of rectilinearity.

3.1.3. Minimizing the error of C_0'. The most satisfactory solution (from the standpoint of error reduction) for minimizing the effect of non-zero values of d(see

section 2.2.2) is to inject sufficient gauging ion into the stream that $C_2 - C_0' \gg d$. Thus, if $(C_2 - C_0') = 100d$, the bias from this source is equivalent to 1 per cent of the discharge. It is desirable, therefore, to know the value of d for each gauging so that C_1 may be suitably chosen. In general, d may vary from river to river, site to site, and time to time. At present, there appears to be little published information on the values of d for particular ions, and each gauging should, therefore, include an estimation of d, at least until there is sufficient confidence that d will be negligible.

If d is not known for a particular gauging, it will probably often be reasonable to assume that d will not be larger than C_0'; approximate knowledge of C_0' will thus allow calculation of suitable values for C_1 and the rate of injection. It is worth noting that the coefficent of variation of analytical results often improves with increasing concentration so that the use of greater values of C_1 and/or the rate of injection may also lead to more precise results.

The approach recommended in the two preceding paragraphs is preferred whenever possible but it may sometimes require impracticably large rates of injection and/or amounts and volumes of chemicals and solutions. The concentration of the gauging ion in the stream may also be subject to limits imposed by health or other considerations. Under these conditions, an alternative approach is first to measure d and to use $(C_2 - C_0' + d)$ in place of $(C_2 - C_0')$ when calculating results. A simple (and probably incorrect) assumption that could be made for this approach is that systematic changes in the apparent concentration of the gauging ion near the injection site are reproduced exactly some time later at the downstream site. A value for d could then be obtained before the gauging by analysis of samples taken at the two sites at times corresponding to the same "plug" of stream water. Such a value for d, together with a series of background water and downstream samples during a gauging, would then allow some compensation for systematic changes in C_0'. Further development of this approach is possible but is considered unnecessary here in view of the uncertain nature of the basic assumption.

3.2. Minimizing random errors

The factors affecting the random error of discharge results are shown by equation 12 (section 2.3) under the condtion that $C_1' \approx C_2$. These factors are discussed below.

3.2.1. Optimum value of C_2. Analytical systems give results whose standard deviation can usually be adequately represented (up to a limit C_L) by an equation of the form:

$$\sigma_a = a + bC$$

where a and b are constants. Thus :

$$R_a = 100\left(\frac{a}{C} + b\right)$$

and the coefficient of variation decreases with increasing concentration until a minimum value of $100[(a/C_L) + b]$ is obtained. Equation 12 shows that both the other two terms involving C_2 decrease as C_2 increases. A further aspect is that calibration curves tend to depart from rectilinearity at higher concentrations. The optimum value of C_2 (from the standpoint of reducing random error) can, therefore, be chosen only on the basis of detailed knowledge of any particular conditions.

The best general guidance that can be given is to make C_2 as large as possible consistent with maintaining (1) a non-increasing value of $R_a\ (C_2)$ and (2) a rectilinear calibration curve. If, for this value of C_2, one or both of the terms $C_2/(C_2 - C_0')$ and $\sigma_s^2\ /\sigma_a^2\ (C_2)$ are appreciably greater than unity, it may be beneficial to use a still larger value of C_2 provided condition (2) is maintained.

3.2.2. Effect of C_0'. Equation 12 shows that increasing concentration and variability of the gauging ion in the stream, C_0' and σ_s^2, worsen the precision of the discharge result. Thus, gauging ions that give the smallest possible values for both parameters are desirable. If the ion employed is such that σ_s^2 is not negligible, it will probably be impossible to vary σ_s^2 at will; the gaugings should then be made at times and positions such that σ_s^2 is minimized.

The use of m separate sampling occasions during one gauging reduces the effect of variations in C_0' but the reduction is, at most, by a factor of m and therefore becomes increasingly less useful with increasing m. However, all other things being equal, it is desirable to use values of m as large as practicable (but also see section 3.2.3) for these will provide some averaging of any short-term variations in stream discharge.

3.2.3. Precision of analytical results. Equation 9 shows that both C_1' and C_2 are of equal importance in affecting the precision of discharge results (when $C_1' \approx C_2$). It is, therefore, most efficient to make the same number of determinations for each type of sample.

Equation 12 shows the intuitively obvious result that the precision of gauging results depends on the coefficient of variation given by the analytical system. The effect of analytical imprecision may be reduced by making n replicate analyses on each sample. However, when $[\sigma_s^2/\sigma_a^2(C_2)] \ll 1$, there is no point in analysing m samples of each type in order to reduce random error. It will be more convenient and just as efficient to make mn replicate analyses on one sample of each type. The numerical values for m and n should be chosen so that the desired precision for the mean discharge results is obtained. It should be emphasized, however, that the effects of systematic variations in C_0' during a gauging may be at least partially eliminated by making $m > 1$ (see section 3.1.3).

3.2.4. Precision of results for the discharge. Statistical confidence limits for $\bar{\bar{Q}}$ can be evaluated from equation 12 for any conditions of interest but no attempt is made here to calculate $R(\bar{\bar{Q}})$ for a range of values of all the factors. It is useful, however, to estimate the best possible precision by putting $C_2/(C_2 - C_0') = 1$ and by neglecting $\sigma_s^2/\sigma_a^2(C_2)$. Equation 12 then reduces to:

$$R(\bar{\bar{Q}}) \approx 1.41\ R_a(C_2)/\sqrt{mn}$$

Thus, if the analytical results follow the Gaussian distribution, 95 per cent confidence limits for $\bar{\bar{Q}}$ will be $= 2.77\ R_a(C_2)/\sqrt{mn}$, and numerical values for these limits are given in Table 1.

This table illustrates only the best that can be achieved. Confidence limits for a practical gauging should be evaluated from the replicate results within the gauging itself. In practice, $C_2/(C_2 - C_0')$ may be greater than 1, $\sigma^2/\sigma_a^2(C_2)$ may not be negligible, only an estimate of the true coefficient of variation will be obtained, and the results may not follow the Gaussian distribution. All these points will lead to

Table 1. *Theoretical best possible 95 per cent confidence limits for the estimated discharge*

Coefficient of variation of single analytical results (percentages)	95 per cent confidence limits for the discharge as a percentage of the discharge (percentages)		
	$mn = 4$	$mn = 9$	$mn = 16$
0.5	0.7	0.5	0.3
1.0	1.4	0.9	0.7
2.0	2.8	1.8	1.4
3.0	4.2	2.8	2.1
4.0	5.5	3.7	2.8

greater numerical confidence limits than those in Table 1. Therefore, analytical systems capable of giving a coefficient of variation no worse than 2 per cent are necessary if 95 per cent confidence limits for the discharge of 2 to 3 per cent are to be obtained without a cumbersome amount of replication.

4. CONCLUSIONS

The considerations above have brought out several desirable aspects of the gauging ion and the analytical system, and both will be chosen appropriately for the application of interest. When the ion and method of analysis have been chosen, the following sequence of actions is recommended for any stream gauging though previous information or experience may allow some of these actions to be omitted or curtailed.

(a) Define the desired accuracy. Decide the accuracy (sum of random and systematic errors) required for the estimated discharge and choose the confidence level to be used to define random errors.

(b) Select the injection and downstream sampling sites. Choose these sites so that complete mixing is obtained, and so that negligible losses or gains of the gauging ion occur (see sections 2.2.3 and 3.1.1(b)). Define any local precautions required to prevent important contamination of samples of the stream during collection and handling in the field.

(c) Check the values of d, C_0 and σ_s^2. Collect at least five samples of the stream from each of the two sites over several hours in pairs corresponding to the same "plug" of stream water. Analyse each sample for the gauging ion and estimate d, C_0 and σ_s^2 from the results (see sections 3.1.3. and 3.2.2).

(d) Decide the concentration and rate of injection of the gauging ion. Choose a suitable value for C_2 (see sections 3.1.3 and 3.2.1). From approximate knowledge of the discharge, choose the concentration and rate of injection of the concentrated solution of the gauging ion. Also choose the dilution of this solution required to ensure $C_1' \approx C_2$, and decide whether or not the dilution is to be made during the gauging (see section 3.1.1(a)).

(e) Decide the degree of replication required. From equation 12 (section 2.3), choose values for m and n such that adequately small random error will be obtained (see sections 3.2.2., 3.2.3 and 3.2.4). Also choose the number of background samples to be collected during the gauging (see sections 3.1.3 and 3.2.2).

(f) Carry out the gauging. In general, before commencing to inject the gauging ion, collect several samples at the injection and downstream sites to obtain estimates of d, C_0 and σ_s^2 for the gauging. (This precaution will usually be unnecessary when C_2 has been chosen so that it is much greater than d and C_0).

Check the rate of injection at intervals during the gauging and ensure that the rate remains adequately constant.

(g) Analyse the samples. Check the linearity of the calibration curve (see section 3.1.2). Analyse all samples alternating downstream samples and the diluted concentrate and evaluate the results to give the ratio $[(C_1' - C_0')/(C_2 - C_0')]$ (see section 3.1.2).

(h) Calculate the stream discharge. Calculate the discharge using equation 3 (section 2.1) making allowance for d if necessary (see section 2.2.2). Calculate the confidence limits (at the chosen confidence level) of the mean discharge from the experimental results.

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BIBLIOGRAPHY / BIBLIOGRAPHIE

1. American Public Health Association, *et al.* 1965. *Standard Methods for the Examination of Water and Wastewater.* 12th edition, p. 166, New York.
2. Bennett, C.A. Franklin, N. L. 1954. *Statistical Analysis in Chemistry and the Chemical Industry,* p. 51. London, Chapman and Hall.

DISCUSSION

Carter: The paper states that it is impossible to determine the value of C_0. If samples of the stream water are taken at the injection site and at the downstream site just before the injection is made, could not the value of C_0 be determined with a great degree of accuracy?

Wilson: Only during the gauging samples can be obtained upstream of the injection site. Under those conditions there is no direct check during a gauging of the background concentration at the downstream sampling site.

Carter: But if samples are taken at the downstream site prior to the injection, then you would also know C_0 at the downstream site.

Wilson: That is assuming that C_0 does not change with time.

Steppuhn: Many difficulties with monitoring background tracer concentrations are avoided in the tracer sampling technique by gelatine-coated film, described in a previous paper.

Wilson: The application of the different techniques depends very much on the methods of analysis used. Colorimetric methods of determination, particularly in very small concentrations, may suffer from the presence of very small concentrations of suspended materials.

II.3 Measuring structures
Ouvrages servant aux mesures

Adverse-bottom-slope weir and orifice

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Abstract. The boundary conditions of an adverse-bottom-slope weir and orifice differ from the standard conditions. This study was undertaken to determine the coefficients of discharge and their variations due to change in the adverse slope.

The solution for the discharge function obtained involves a combination of dimensional analysis and laboratory experiments. In all cases of adverse-bottom-slope weir or orifice investigated in this study, the discharge for the same head was higher than in the case of horizontal-bottom ones. The difference increases with increase of the slope and attains its maximum value at a slope around 2.5: 1 for a weir and 4: 1 for an orifice.

The increase in the discharge coefficient obtained was 16 and 9 per cent for adverse-bottom-slope weir and orifice respectively. These results suggest the use of adverse-bottom-slope weir and gates instead of horizontal-bottom standard ones as they have a higher coefficient of discharge for the same head.

DEVERSOIR ET ORIFICE A FOND A PENTE CONTRAIRE

Résumé. Les conditions limites qui sont celles d'un déversoir ou d'un orifice à fond à pente contraire sont différentes de celles qui existent habituellement. L'objet de cette étude est de déterminer leur coefficient de débit et ses variations en fonction des variations de la pente contraire.

La solution de la fonction de débit obtenue nécessite de combiner analyse dimensionnelle et expérimentation en laboratoire. Dans tous les cas de déversoir, ou d'orifice, à fond à pente contraire utilisés dans cette étude, le débit pour une même hauteur de charge est plus grand pour les fonds à pente contraire que pour les fonds horizontaux. La différence décroît avec la diminution de la pente et atteint sa valeur maximale pour une pente de 2,5 : 1 pour un déversoir et de 4 : 1 pour un orifice.

L'accroissement du coefficient de débit obtenu est de 16 % pour le déversoir et de 9 % pour l'orifice à fond à pente contraire.

Ces résultats invitent à utiliser des déversoirs et des orifices à fond à pente contraire, car ils ont un coefficient de débit plus élevé pour une même hauteur de charge.

ESTRUCTURAS DE MEDICIÓN. VERTEDERO Y ORIFICIO CON CONTRAPENDIENTE

Resumen. La foma límite de un vertedor y orificio con contrapendiente es diferente a los de condiciones normales. El objeto del presente estudio es determinar su coeficiente de caudal y la variación debida al cambio de la contrapendiente. La solución para la función de caudal obtenida incluye una combinación de análisis dimensional y experimentos en laboratorio. En todos los casos de vertederos u orificios con contrapendiente investigados en el presente estudio, el caudal para la misma altura era mayor para la contrapendiente que en el caso de fondo horizontal. La diferencia aumenta con el crecimiento de pendiente y alcanza su máximo valor con una pendiente próxima a 2,5:1 para los vertederos y a 4:1 para los orificios.

El aumento obtenido en el coeficiente de caudal fue de 16 % y 9 % para contrapendiente en vertedero y orificio respectivamente. Dichos resultados sugieren el empleo de vertederos y compuertas con contrapendiente en lugar de los normales horizontales, puesto que poseen un mayor coeficiente de caudal para la misma altura.

ВОДОСЛИВ И ОТВЕРСТИЕ С ОБРАТНЫМ УКЛОНОМ ДНА

Резюме. Граничная форма водослива и отверстия с обратным уклоном дна отличаются от стандартных условий. Целью этого исследования являлось определить коэффициент расхода этих гидрометрических устройств и его изменение, обусловленное изменением в обратном уклоне.

Полученное решение для формулы расхода включает сочетание анализа размерностей и лабораторные эксперименты. Во всех случаях обратного уклона дна водослива или отверстия, рассмотренных в этой работе, расход для одного и того же напора оказался выше, чем в случае горизонтального дна. Разница увеличивается с увеличением уклона дна и достигает максимального значения при уклоне около 2,5 : 1 для водослива и 4 : 1 для отверстия.

Полученное увеличение в коэффициентах расхода составило 16 % и 9 % соответственно для водослива и отверстия. Эти результаты позволяют рекомендовать применение водосливов и отверстий с обратным уклоном дна вместо стандартных горизонтальных, поскольку они дают более высокие значение коэффициентов расхода при том же самом напоре.

INTRODUCTION

The value of the discharge coefficient is determined by the boundary conditions and the fluid characteristics. A change in the geometry of a structure may appreciably alter the discharge coefficient.

If the bottom of the channel upstream from the weir crest or orifice plate rises in the direction of the flow, the bottom will be described as an adverse-bottom slope. In such a case, the normal coefficient of discharge for the weir or orifice is appreciably altered.

Unfortunately, little research has been devoted to weirs or orifices with adverse-bottom slopes. This study was undertaken to determine the variations in discharge coefficients for weirs and orifices due to changes in the adverse slope of the bottom, by re-examination of published data on weirs and by analysis of published data for orifices (Abaza 1964).

ANALYTICAL DEVELOPMENT

The discharge coefficient for a weir or orifice is closely related to the geometry of the boundary expressed by the Euler number, the characteristics of the flowing fluid expressed by the Reynolds and Weber numbers, and the effect of gravity expressed by the Froude number.

To simplify the problem in the present study, the flow was assumed to be two-dimensional. A sharp-crested weir and a rectangular sharp-edged, full-width orifice were used.

Dimensional analysis

As the gravitational flow pattern for the asymmetrical sharp-crested weir or rectangular orifice has not yet been subjected to exact mathematical description, the direct solution for the discharge function will be made using a combination of dimensional analysis and experimentation. The procedure used is similar to that employed by Kindsater (1957) and Dai-Ly (1962).

The discharge functions for adverse-slope weir and orifice were expressed respectively as (1) :

$$C_A = f\left(\frac{H}{P}, \frac{X}{P}, R, W\right) \qquad \text{weir} \quad 1(a)$$

$$C_A = f\left(\frac{h}{P}, \frac{b}{P}, \frac{X}{P}, R, W\right) \qquad \text{orifice} \quad 1(b)$$

in which C_A represents the discharge coefficient, H, W, P. are as shown in Figure 1, and R and W represent Reynolds and Weber numbers.

Significance of the boundary conditions

The ratio X/P indicates the influence of the bottom slope. The ratio $h/(h+P)$ and H/P are a measure of the contraction characteristics and are complementary to h/P as a contraction ratio.

Significance of fluid characteristics

Little is known about the separate influence of viscosity, represented by R, and surface tension, represented by W in equation 1. It is known, however, that the relative influence of the two fluid properties decreases as the head on the orifice or weir increases, while both surface tension and density stay constant. Both R and W can then be expressed in terms of h, H and b, and the relative influence of the combined fluid properties diminishes as either H, h or b becomes relatively large. In such a case the relationship between the coefficient of discharge and the principal boundary form ratio can be shown to be

$$C_A \text{ (weir)} = f\left(\frac{H}{P}\,;\ \frac{X}{P}\right) \qquad 2(a)$$

and

$$C_A \text{ (orifice)} = \left(\frac{h}{h+P}\,;\ \frac{b}{P}\,;\ \frac{X}{P}\right). \qquad 2(b)$$

If we denote C_H as the coefficient of discharge for horizontal-bottom slope, it will represent the case for $X/P \to \infty$ and the same relationship still stands.

The ratio C_A/C_H of the discharge coefficient for the adverse-bottom-slope weir or orifice, C_A, to the discharge coefficient for horizontal-bottom-slope weirs or orifice, C_H, will be used in the analysis as a dimensionless ratio. This will eliminate the parameter H/P for the weir and $h/(h+P)$ for the orifice. Similarly the effect of the velocity distribution in the approach channel will be minimized. The analysis which follows will be in terms of percentage change

$$[(C_A/C_H) - 1] \cdot 100.$$

RESULTS AND DISCUSSION

Weirs

Data given by Bazin (1892), Dai-Ly (1962) and the United States Bureau of Reclamation (1948) are plotted for comparison in Figure 2. The approximate maximum increase $[(C_A/C_H) - 1] \cdot 100$ (for $X/P = 2.0$) was 14 per cent for Bazin and 16 per cent (for $X/P = 2.50$) for Dai-Ly.

From the figure it is clear that the slope of the curve, or the rate of increase of the percentage $[(C_A/C_H) - 1] \cdot 100$ increases until the curves attain their maximum at the indicated ranges. The USBR data did not attain their maximum in the available range although they follow the same pattern of change. This suggests the use of an adverse-bottom-slope weir of a length 2.0 to 2.50 times its height. This slope would give an increase in discharge of about 16 per cent for the same head.

Orifice (gates)

From analysis of the data, it was found that the ratio C_A/C_H is not affected by the ratio $h/(h+P)$ and consequently is independent of h, which means that the significant parameters will be X/P and b/P.

The variation in the percentage $[(C_A/C_H) - 1] \cdot 100$ versus different values of X/P are plotted in Figure 3 with b/P as a parameter. Figure 4 shows values of the percentage versus b/P with X/P as a parameter. In Figures 2 and 3, three main curves corresponding to $X/P = 0.62$, 1.25, and 2.50 are plotted. By interpolation, a family of curves with different values of X/P ranging from 0.3 to 2.50 are drawn.

In Figure 3, the curves show a rapid rate of increase in percentage as X/P begins to increase from the vertical position. As X/P increases the slope of the curves, the rate of increase in the percentage decreases rapidly. Although the range of experimentation does not cover the point where the percentage is maximum, the trend of the curves shows that all curves will attain their maximum value at a value of X/P equal to approximately 4.0 where the increase in the percentage $[(C_A/C_H) - 1] \cdot 100$ would reach 7 to 8 per cent. This suggests the use of gates with an adverse-bottom slope. This would give an increase of about 7 per cent in discharge for the same head.

CONCLUSIONS AND RECOMMENDATIONS

In all cases of weirs or orifices (gates) with adverse-bottom slopes investigated in the present study, the discharge was greater than in cases of corresponding weirs or orifices with horizontal-bottom slopes. The entrance conditions and the change in velocity distribution in the approach channel effect on the discharge coefficient is minimized by using the percentage of increase $((C_A/C_H) - 1) \cdot 100$.

For an orifice with a constant value of b/P, the percentage of increase in the discharge coefficient increases with the increase in X/P. Increasing b/P (by increasing b) for the same value of X/P, the percentage increases and reaches its highest value (within the available range) for $b/P = 0.37$.

The maximum percentage increase in the discharge coefficient obtained for the adverse-bottom-slope weir was 16. The maximum increase for the adverse-bottom-slope orifice was 7 per cent. This leads us to suggest the use of weirs and orifices with

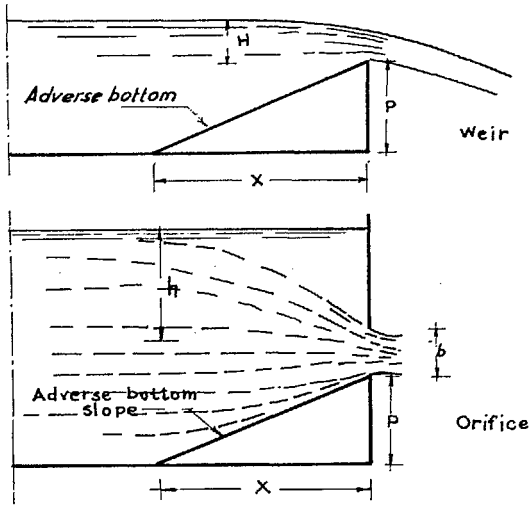


Figure 1. *Sketch of rectangular adverse-bottom slope weir and orifice*

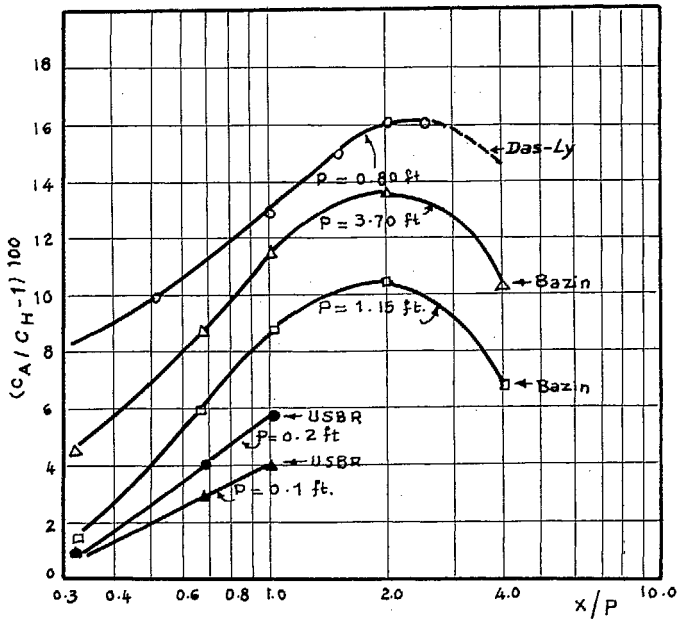


Figure 2. *Percentage of increase in discharge coefficient for sharp-crested weir, caused by adverse-bottom slope, versus X/P from different sources of data*

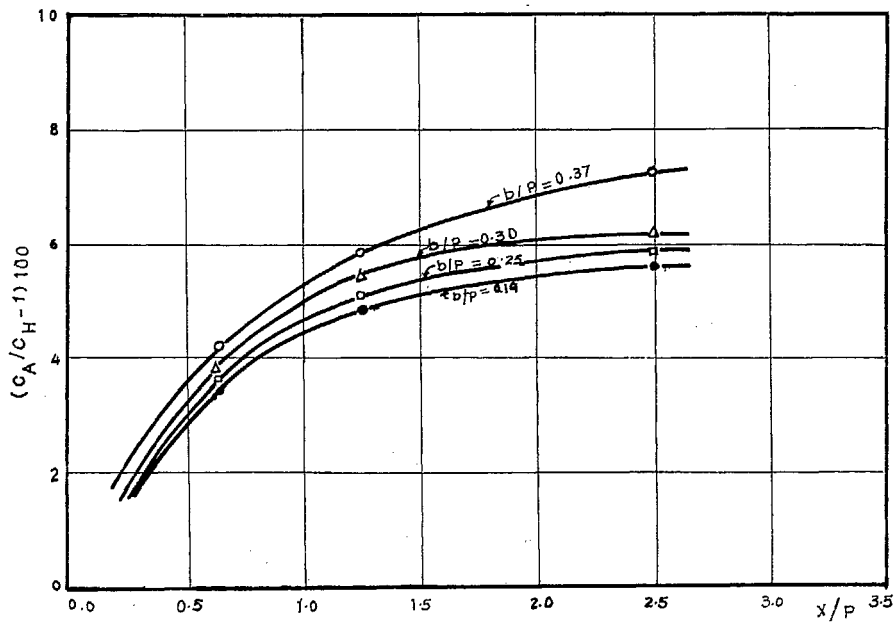


Figure 3. *Percentage of increase in discharge coefficient for rectangular orifice, caused by adverse-bottom slope, versus X/P*

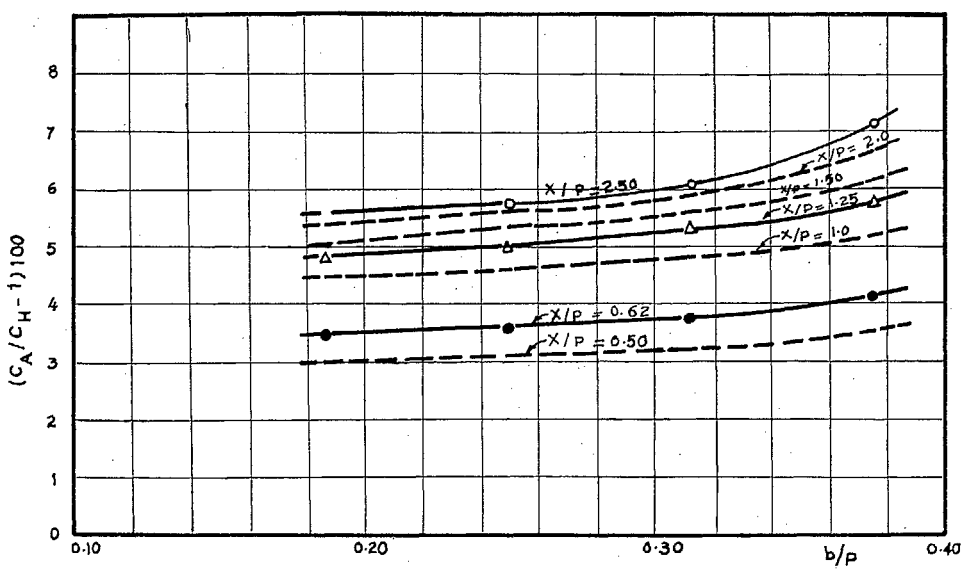


Figure 4. *Percentage of increase in discharge coefficient for rectangular orifice, caused by adverse-bottom slope, versus b/P*

adverse bottom-slopes instead of a horizontal bottom to gain an increase in discharge for the same head.

The range of 0.0 to 2.5 for X/P will be sufficient for most field applications of weirs or orifices with adverse-bottom slopes but, as a matter of academic interest, more study beyond this range is suggested. For the orifice, ratios beyond $b/P = 0.37$ are needed which require an extension of these dimensionless relationships to other ranges. A study of the adverse-bottom-slope effect equivalent to the effect of virtual vertical displacement of the horizontal bottom for both weir and orifice is also suggested.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Abaza, M. I., 1964. Coefficient of discharge for orifices and weirs with adverse-bottom slopes. Master's thesis. Utah State University, Logan, Utah.

Bazin, H. E. 1892. Experiments on inclined surface weirs. *Engineers's Club of Philadelphia*, vol. 9, no. 3, p. 231-44.

Dai-Ly. 1962. Coefficient of discharge for a silted weir. Master's thesis. Utah State University, Logan, Utah.

Kindsvater, Carl E.; Carter, Roland W. 1957. Discharge characteristics of rectangular thin-plate weirs. *Proceedings A.S.C.E.*, paper 19, 1453.

King, H. W. 1954. *Handbook of Hydraulics.* New York, Mc-Graw Hill. United States Bureau of Reclamation. 1948. Studies of crests for overfall dams. Part II: Hydraulic investigations. *Bulletin 3.*

Laboratory calibration of the Walnut Gulch supercritical flow-measuring flume

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Abstract. A new supercritical measuring flume is being used to gauge sediment-laden ephemeral flows in steep channels. The transition from the natural channel to the straight modified trapezoidal measuring section of the flume consists of a cylindroid surface. The flume is kept free of deposition by a V-shaped floor which slopes in the direction of flow. The head is measured at the midpoint of the straight section.

Ten of these concrete flumes have been installed in the Walnut Gulch Watershed near Tombstone, Arizona. Eight of the flumes have already been calibrated with models in the laboratory. The largest has a bottom width of 36.6 metres and a capacity of about 740 m^3/sec. This structure is the largest known precalibrated flume now in operation.

The design of the flumes, the laboratory calibration data and some observations of the field operation are discussed.

ETALONNAGE EN LABORATOIRE DU CANAL DE MESURE DU DEBIT SUR-CRITIQUE A WALNUT GULCH

Résumé. Un nouveau canal de mesure surcritique sert actuellement à mesurer les cours d'eau chargés de sédiments dans des canaux escarpés. La transition entre le canal naturel et la section de mesure qui est rectiligne et a la forme d'un trapèze modifié s'effectue par une surface cylindroïde. On évite le dépôt de sédiments dans le canal au moyen d'un fond en forme de V incliné dans la direction du courant. La hauteur d'eau est mesurée au milieu de la section droite.

Dix de ces canaux en béton ont été installés dans le bassin hydrographique Walnut Gulch près de Tombstone (Arizona). Huit de ces canaux ont déjà été étalonnés avec des modèles de laboratoire. Le plus grand a une largeur au fond de 36,6 mètres et une capacité d'environ 740 m^3/s. Cette structure est, croit-on, le plus grand canal précalibré actuellemnt en service.

Le dessin des canaux, les données de l'étalonnage en laboratoire et quelques observations sur le fonctionnement sur le terrain font l'objet d'un examen critique.

CALIBRACIÓN EN LABORATORIO DEL EMBUDO DE WALNUT GULCH PARA LA MEDICIÓN DE FLUJO SUPERCRÍTICO

Resumen. Se está utilizando un embudo de medición supercrítico a fin de aforar corrientes efímeras cargadas de sedimentos en cauces de fuerte pendiente. La transición del cauce natural a la sección recta trapezoidal modificada del embudo está formada por una superficie cilindroide. El embudo se mantiene libre de depósitos gracias a un fondo en forma de V con pendiente en la dirección del flujo. La altura o carga se mide en el punto medio de la sección recta.

En el río Walnut Gulch, cerca de Tomstone, Arizona, se han instalado diez de dichos embudos de hormigón. Ocho de los mismos han sido ya calibrados con modelos en laboratorio. El mayor tiene una anchura de fondo de 36,6 m y una capacidad de unos 740 m^3/seg. Esta estructura es la mayor de embudo precalibrado conocida actualmente en servicio.

El diseño de los embudos, los datos de calibración en laboratorio y algunas observaciones en campo, están siendo ahora estudiadas y discutidas.

ЛАБОРАТОРНАЯ ТАРИРОВКА СВЕРХКРИТИЧЕСКОГО ЛОТКА ДЛЯ ИЗМЕРЕНИЯ СУММАРНОГО РАСХОДА НА ВОДОСБОРЕ «ВОЛНУТ ГУЛЬЧ»

Резюме. Новый измерительный лоток со сверхкритическим режимом потока используется для измерения кратковременных потоков с обильным содержанием наносов в руслах с большими уклонами. Переход от естественного русла к прямолинейному модифицированному трапецоидальному измерительному сечению лотка состоит из цилиндрической поверхности. Лоток поддерживается свободным от отложения наносов благодаря V-образной форме поперечного сечения, имеющей уклон в направлении течения. Напор измеряется в средней точке прямолинейного участка.

Десять таких бетонных лотков были построены на водосборе «Волнут Гульч» близ Томбстона, штат Аризона. Восемь из них уже протарированы с помощью моделей в лаборатории. Наибольшее сооружение имеет ширину дна 36,6 м и пропускную способность около 740 м³/сек. Это сооружение является наибольшим среди известных предварительно протарированных лотков, находящихся в эксплуатации.

Приводятся данные о конструкции лотка, лабораторной тарировке и некоторые наблюдения по эксплуатации в полевых условиях.

INTRODUCTION

A new type of flume was developed to measure the ephemeral, flashy and sand- and gravel-laden flows conveyed by the steep streams in the Walnut Gulch watershed near Tombstone, Arizona, U.S.A. The use of weirs, a velocity-area method using a current meter or chemical dilution methods to obtain a continuous record of water yield from watersheds in this area was impractical or impossible. The flume discussed in this report can cope with the field conditions and obtain a measurement. It has been previously described by the writer[1] and the field performance has been reported by Osborn, Keppel and Renard.[2]

Analyses of data obtained from laboratory calibrations for eight of these flumes have yielded prediction equations for the head-discharge relationship. These new analyses will be described along with observations of flume performance.

FLUME DESIGN

All flumes have a curved entrance approach (4.572 metres long) to a straight portion (6.096 metres long) that has a shallow V-shaped floor and side walls of 1 on 1 slope. Figure 1 shows the general design of a typical flume.

The length of the straight portion should be about twice the maximum head.[1] The curved approach has a cylindroid surface defined by the equation:

$$y = 0.03x + \frac{z - 0.3229x^2}{0.0287x^2 + 1} \qquad (1)$$

1. Gwinn, W. R. Walnut Gulch supercritical measuring flume. *Transactions, American Society of Agricultural Engineers*, vol. 7, no. 3 (1964), pp. 197-99.
2. Osborn, H. B.; Keppel, R. V.; Renard, K. G. *Field performance of large critical-depth flumes for measuring runoff from semiarid rangeland.* United States Department of Agriculture, Agricultural Research Service, ARS 41-69, March 1963.

in which x = horizontal co-ordinate (m) positive in the upstream direction; y = vertical co-ordinate (m); and z = horizontal co-ordinate (m) normal to the centreline of the flume (see Figure 1 for the origin of the co-ordinates). A perspective view of this surface is shown in Figure 2.

All the flumes have the same curved entrance approach, are 10.67 metres long, and differ only in width (W) and floor cross-slope (S_V). The width selected for a flume depended upon discharge rate, available head and on the need to obtain a contraction ratio (measuring section flow area divided by the approach channel flow area) of 0.5 or less. Dimensions of the flumes are given in Table 1.

Table 1. *Summary of Walnut Gulch flumes that have been calibrated in the laboratory*

Flume No.	Floor cross-slope (S_v)	Flume width (W) (m)	Maximum discharge (m^3/s)	Model length scale	Data on figure number
1	15	36.58	736	1:40	4
3	7.5	9.14	170	1:32	5
4	10	1.52	34	1:30	6
6	10	21.34	467	1:30	7
7	10	12.19	243	1:30	8
8	10	12.19	243	1:30	9
11	10	9.14	170	1:30	10
15	10	12.19	235	1:30	11

The structures have a 3 per cent slope in the downstream direction to ensure movement of sand and gravel through the flumes. The inverts of the flumes were placed about 0.6 metres above the streambed to reduce backwater on the flume. The head is measured at the midpoint of the straight portion of the flume. Velocities are supercritical at the head measuring section. Therefore, the structure is known as the Walnut Gulch supercritical flow-measuring flume.

MODEL STUDIES

Detailed topographic data were obtained for a distance of about 300 metres upstream and 100 metres downstream from each site. The location, density and size of any vegetation in the channel were determined and the nature of the bed material noted. This information was needed for construction of a model of the site.

Selection of the model scale was based on the size of the area to be modelled, laboratory basin, water supply and the approximate roughness of the field channel. Model scales used are given in Table 1. The topographic details of the approach channel were carefully modelled in the laboratory test basin with Portland cement mortar. Vegetation was simulated in the model with screen wire selected for density and properly located in the channel. The flume was constructed of redwood and varnished to produce a smooth surface. Flush mounted piezometers were installed at several points in the bed of the approach channel to obtain data needed for calculating the friction factor.

Approximately 50 test flows were used in calibrating each variation of the particular model. Pressures, discharge rate, water temperature, and general flow conditions were recorded for each test.

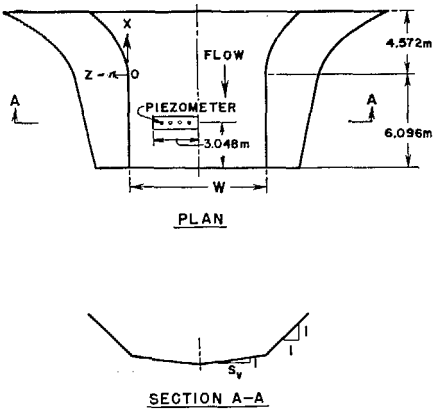


Figure 1. *Walnut Gulch supercritical flow-measuring flume*

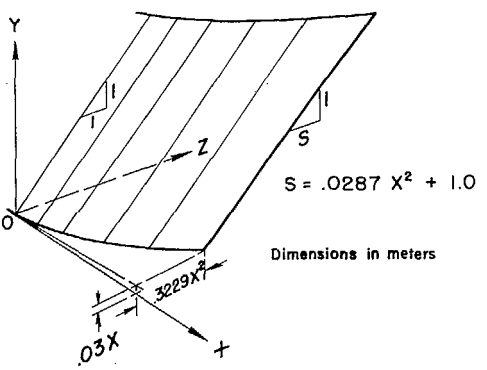


Figure 2. *The approach cylindroid surface*

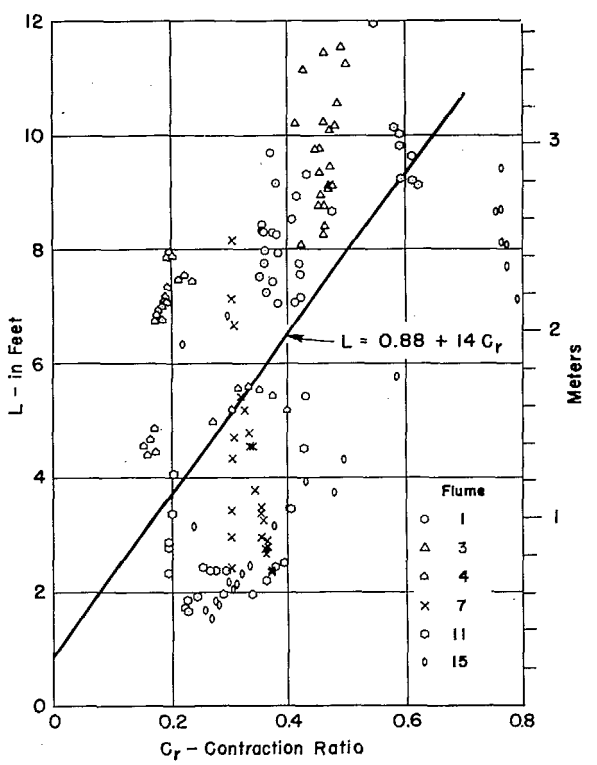


Figure 3. *Computed contraction ratio, C_r, and horizontal length, L, between critical section and point of head measurement for model ratings of Walnut Gulch flumes*

DATA ANALYSIS AND FLUME THEORY

Critical discharge occurs in the straight portion of the flume and is computed by the following equation:

$$Q_c^2 = \frac{gA_c^3}{\alpha_c T_c} \tag{2}$$

in which Q_c = discharge; g = acceleration due to gravity; A_c = cross-sectional area of flow; α_c = velocity distribution coefficient; T_c = top width of flow; and c = subscript relating to the critical section.

The total specific energy, H_p, above the flume zero at the point of head measurement, assuming no energy loss between the critical section and the head measuring section, would be:

$$H_p = h_c + \frac{\alpha_c V_c^2}{2g} + 0.03L \tag{3}$$

in which h_c = depth of flow above V-shaped floor at the critical section; V_c = velocity at critical section; L = horizontal length between critical section and point of head measurement; and p = subscript relating to the point of head measurement. Also,

$$H_p = h_p + \frac{Q_c^2}{2gA_p^2} \tag{4}$$

Subtracting equation 4 from equation 3 and using equation 2 and $Q = AV$, yields the relation

$$h_p = h_c + \frac{A_c}{2T_c} + 0.03L - \frac{Q_c^2}{2gA_p^2}. \tag{5}$$

Values of L were computed by an iterative technique using the test values for h_p and Q and an assumed value (chosen low) for the initial value of h_c. The contraction ratio, C_r (A_p divided by the approach channel flow area) was computed at the same time. the values of L were plotted against the corresponding values of C_r on Figure 3. The following straight-line equation was fitted to the data:

$$L \text{ (in feet)} = 0.88 + 14\, C_r \tag{6}$$

$$L \text{ (in metres)} = 0.29 + 4.59\, C_r. \tag{7}$$

The discharge coefficient, C, was calculated for each test flow from the formula:

$$Q = C\frac{T_p}{2}\sqrt{2g}\, h_p^{1.5}. \tag{8}$$

These C values are plotted against the corresponding values of head, h_p, for each flume in Figures 4 to 11.

A computed rating, h_p versus Q, is obtained from equation 5 by iteration. A value of h_c is selected and Q_c is calculated from equation 2. A value for L is assumed and h_p is calculated from equation 5. A backwater computation is made from the point

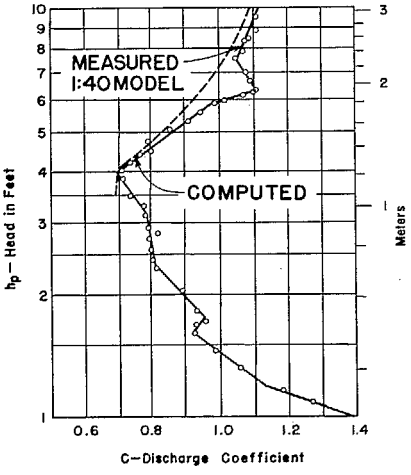


Figure 4. *Discharge coefficients for Walnut Gulch Flume No. 1*

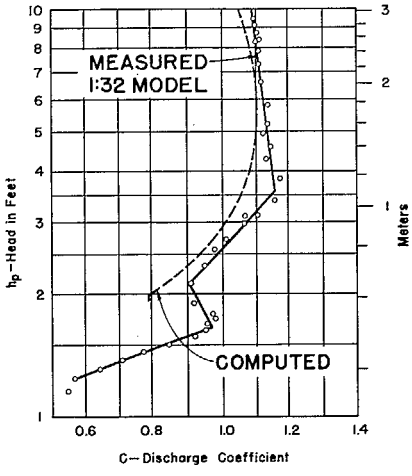


Figure 5. *Discharge coefficients for Walnut Gulch Flume No. 3*

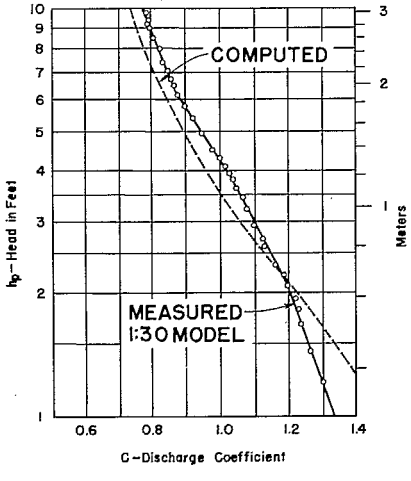


Figure 6. *Discharge coefficients for Walnut Gulch Flume No. 4*

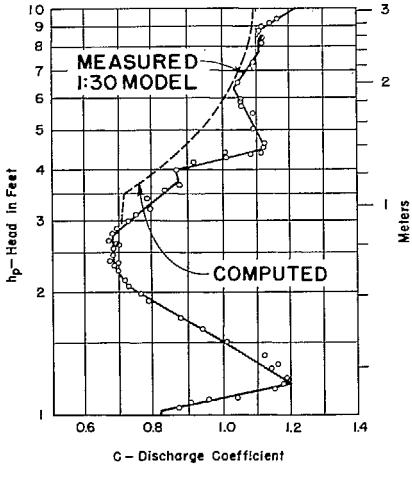


Figure 7. *Discharge coefficients for Walnut Gulch Flume No. 6*

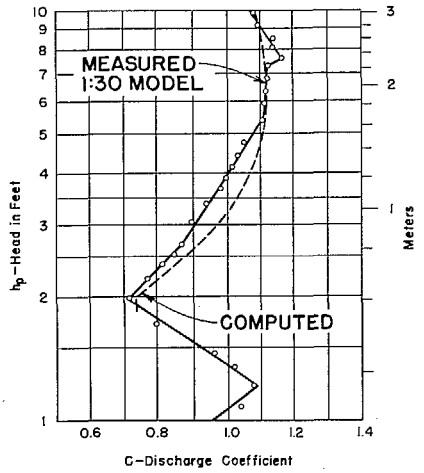


Figure 8. *Discharge coefficients for Walnut Gulch Flume No. 7*

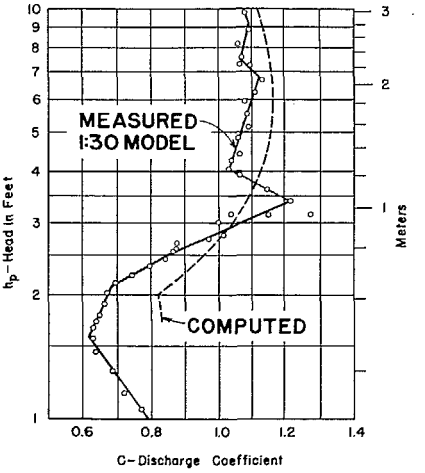


Figure 9. *Discharge coefficients for Walnut Gulch Flume No. 8*

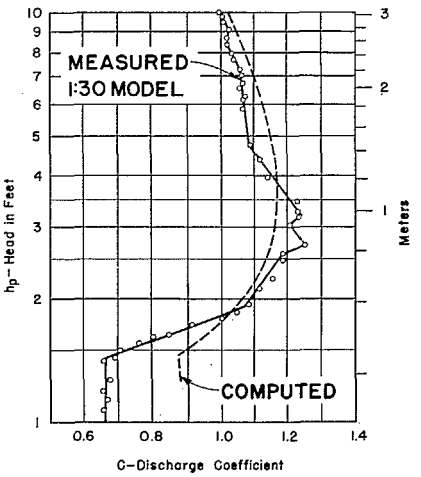


Figure 10. *Discharge coefficients for Walnut Gulch Flume No. 11*

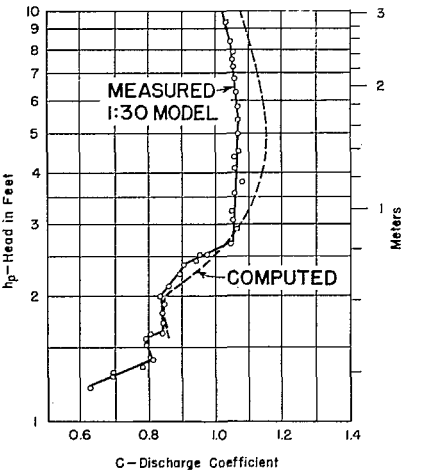


Figure 11. *Discharge coefficients for Walnut Gulch Flume No. 15*

of head measurement to a point 30 metres upstream from the flume. This estimates the water surface elevation and the cross-sectional area of the approach channel. The contraction ratio, C_r, is calculated and used to compute the value of L in equation 7. If L does not agree with the initially assumed value for L, the calculation is repeated. This iteration is continued until a balance is obtained. The C values computed from the calculated Q and h_p values are plotted as dashed lines on Figures 4 to 11 and labelled "computed". Solid lines are used to connect the measured C value points.

Values of Manning's n were expressed in prototype scale by applying a multiplier equal to the one-sixth power of the length scale ratio to the model values of n. They ranged from 0.02 to 0.04 (within estimated values for the prototype) for the finished mortar surface with simulated brush in the model.

OBSERVATIONS

An examination of the C versus h_p curves shows good agreement between the measured values and the computed curves. These curves can only be used for the h_p values where the side walls are exerting control over the flow, that is, above the reversal point of the computed C curves. This point on the computed C curve corresponds to the head at which contractions begin within the flume. This head is equal to the vertical distance from the flume zero to the point of intersection of the wall with the floor of the flume. For example, flume 7 (Fig. 8) has a half width of 6.095 metres and a floor cross-slope of 1 on 10. Therefore, there is a rise of 0.6095 metres from the bottom of the V to the intersection of the floor and the wall. This rise corresponds to the value of h_p at the reversal of direction of the computed C line.

The measured C values for flume 6 (Fig. 7) show the greatest departure from the computed values, with the maximum deviation occurring at a head of about 1.4 metres. This is caused by wave formation in the approach due to a sharp bend located upstream of the flume. The flume was constructed before the model studies. Thus, experience would indicate the desirability of a preconstruction model study for these flumes. For those flumes modelled before construction, numbers 3, 4, 7 and 11, the coefficients agree fairly closely (5 per cent or less) with the computed values.

For flows within the floor of the V-section (lower head range), it would be desirable to make field determinations of the C values. These lower flows have a combination of channel and flume control.

The field experience with these flumes has been good and they have remained free of any sediment deposits in the measuring section.

SUMMARY

The Walnut Gulch supercritical flow-measuring flume is used to measure flows in Walnut Gulch and its tributaries near Tombstone, Arizona.

Ratings can be estimated for the higher flow ranges but model tests are required to determine flume ratings in the lower flow ranges. Model tests are also helpful in determining flume alignment if the flume is to be placed below a sharp bend in a stream.

Model results are portrayed in relationships of discharge coefficients to piezometric head in the supercritical flow portion of the flume.

Discharge coefficients computed using the contraction ratio relationship for the location of critical depth satisfactorily predicted the shape of the discharge coefficient versus piezometric head curve for flows controlled by the sides of the flume.

DISCUSSION

Colenbrander: Did you determine the influence of the approach channel on the discharge during the laboratory tests?

Gwinn: These tests have been made. Generally in the construction of a flume an unfilled condition exists. The channel, at first, is not filled, and after a long time the channel will fill upstream of the flumes. In our models we made the test in both conditions, unfilled and the so-called filled condition of each flume. The data presented in the paper correspond to the natural condition, that is the unfilled.

Kornitz: 1. Have submergence conditions been investigated?

2. Measurement of water level at the critical flow means unstable conditions: for the same water level different discharges may occur.

Gwinn: We have set the flumes roughly 0.6 metres above the natural channel bed, in order to avoid at least part submergence. Backwater studies have been carried out. Backwater occurred only on one flume due to a tributary which came in downstream of this flume. We checked it for tail-water-conditions.

Strilaeff: 1. At what point within the weir are water levels recorded?

2. Have these weirs been used on streams with minimal slopes?

Gwinn: 1. The point of the head measuring-section is at the midpoint of the straight section.

2. These are generally 1 per cent channel slopes. The flow in the stream itself is nearly critical in most cases.

Free surface subcritical flow measurement

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Abstract. Previous work reported by the authors has been concerned with submerged (subcritical) flow measurement at open channel constrictions. The methods of analysis developed to date can be applied to numerous types of side and overflow constrictions. A notable example would be bridge constrictions with the so-called "Abnormal Stage Condition", for which very little data are available. The techniques should be applied to the expanding urban hydrology research programme regarding free surface flow measurement at closed conduit constrictions. Stage-discharge ratings for open channel sections could be analysed mathematically and experimentally for the effects of channel geometry and boundary resistance. Changes in the rating curve with time at a particular station could be related to changes in hydraulic characteristics of the channel. Also, the use of such techniques would provide a means of testing the accuracy of historical run-off records.

MESURE D'UN ECOULEMENT A SURFACE LIBRE SOUS-CRITIQUE

Résumé. Dans un précédent ouvrage les auteurs ont discuté la question des mesures d'un écoulement submergé (sous-critique) à surface libre au droit des rétrécissements. Les méthodes présentes d'analyse sont applicables à de nombreux types de rétrécissements latéraux ou de fond. Un exemple notable est fourni par les piles de ponts dans les conditions dites de niveau anormal, pour lesquelles il y a peu de données. Ces techniques pourraient être appliquées aux programmes en extension de recherches en hydrologie urbaine, portant sur les mesures d'un écoulement libre aux rétrécissements des conduites fermées. Les relations niveau/débit pour des sections à écoulement libre pourraient être analysées mathématiquement et expérimentalement en ce qui concerne les effets de la géométrie du canal et la résistance des parois. On pourrait relier les changements des courbes d'étalonnage dans le temps à un endroit précis aux variations des caractéristiques hydrauliques du canal. Ainsi, l'utilisation de telles techniques fournirait un moyen de juger de la précision des relevés historiques de débit.

MEDICIÓN DE FLUJO SUBCRÍTICO EN SUPERFICIE LIBRE

Resumen. El trabajo previamente realizado por los autores trataba de la medición del flujo sumergido (subcrítico) en constricciones de cauce con superficie libre. Los métodos actualmente desarrollados se pueden aplicar a numerosos tipos de constricciones laterales y de exceso de flujo. Un ejemplo notable serían las constricciones de puente con la llamada "condición de nivel anormal", para las cuales existen pocos datos. Estas técnicas se aplican a programas de investigación de hidrología urbana en expansión, para medición de caudal en superficie libre en constricciones de conducción cerrada. Las relaciones nivel-caudal para las secciones de canal abierto se podrían realizar matemáticamente y experimentalmente para localizar los efectos de la geometría de cauce y las limitaciones del método. Los cambios en el tiempo de dicha curva en una estación determinada se podrían relacionar con los cambios en las características hidráulicas del cauce. También con el empleo de dichas técnicas se podría obtener un medio de analizar la precisión de los datos históricos de escorrentía.

ИЗМЕРЕНИЕ ПОТОКА СО СВОБОДНОЙ ПОВЕРХНОСТЬЮ И ДОКРИТИЧЕСКИМ РЕЖИМОМ

Резюме. Предыдущая работа, изложенная авторами, была посвящена измерению расхода воды в сужениях открытых русел с подтопленным (докри-

тическим) режимом. Методы анализа, разработанные к настоящему времени, могут быть применены ко многим типам боковых и полных сжатий. Выдающимся примером должны являться мостовые сжатия с так называемым «ненормальным состоянием уровня», о которых имеется весьма мало данных. Методика должна быть применима к исследовательской программе развивающейся городской гидрологии, имея в виду измерение потока со свободной поверхностьюв сужениях закрытых трубопроводов.

Кривые расходов для участков открытых русел могут быть проанализированы математически и экспериментально с целью учета геометрии русла и граничного сопротивления.

Изменения кривых расходов во времени могут быть связаны с изменениями гидравлических характеристик русла. Кроме того, использование предлагаемой методики должно дать средства для проверки точности записей исторического стока.

INTRODUCTION

The primary emphasis in hydrometry, to date, by the authors has been the analysis of subcritical flow at open channel constrictions. The two flow conditions involved in rating open channel constrictions, either flumes (side constrictions) or weirs (overflow constrictions) are free (critical or modular) flow and submerged (subcritical or nonmodular) flow. The distinguishing difference between the two flow conditions is that critical depth occurs in the throat of the side constriction, or near the crest of the overflow constriction, for the free flow condition. To determine the free flow discharge, a critical-flow control requires only the measurement of a depth upstream from the point of critical depth. When the downstream or tailwater depth is raised sufficiently, the flow depth everywhere in the constriction is greater than critical depth, and submerged flow (drowned) conditions exist. With submerged flow, a change in the tailwater depth also affects the upstream depth and the constriction rating requires that two flow depths be measured, one upstream (y_u) and one downstream (y_d) from the structure. The flow condition at which the régime changes from free flow to submerged flow is a transition state and the value of submergence ($S = y_d/y_u$) at which this condition occurs is referred to as the transition submergence, S_t.

CONSTRICTIONS

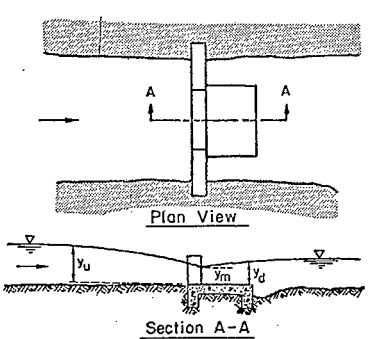
Side constrictions

When dimensional analysis is applied to subcritical flow through a side constriction such as the flume shown in Figure 1(a), three pi-terms are obtained. The first pi-term, π_1, is the Froude number, F_m, evaluated at the section of minimum flow depth, $y_m (F_m = V/(gy_m)^{1/2})$. The submergence, S, elevation, or fall, $y_u - y_d$, divided by the minimum flow depth, y_m, is the third pi-term, π_3. The logarithm of π_2 plotted against π_3 as shown in Figure 1(b) yields the distribution of the lines of constant submergence for the submerged flow calibration curves shown in Figure 1(c) [ref. 6]. The true distribution of the lines of constant submergence is given by the curved line in Figure 1(b) which best fits the data, whereas the straight line is an approximation of the relationship over a large range of submergence. The straight-line relationship in Figure 1(b) has merit because of the approximate submerged flow equation which can be written.

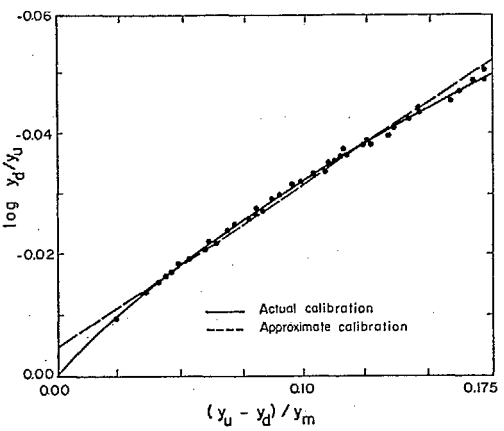
$$Q = \frac{C_2(y_u - y_d)^{n_1}}{[-(\log S + C_3)]^{n_2}} \tag{1}$$

$$Q = C_1 y_u^{n_1}. \tag{2}$$

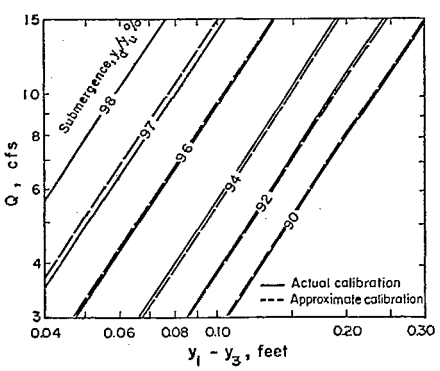
The transition submergence, S_t, can be computed by setting equations 1 and 2 equal to each other and solving for S by trial and error. The value of S_t on the logarithmic submerged flow calibration can then be used as the free flow calibration as shown in Figure 1(d).



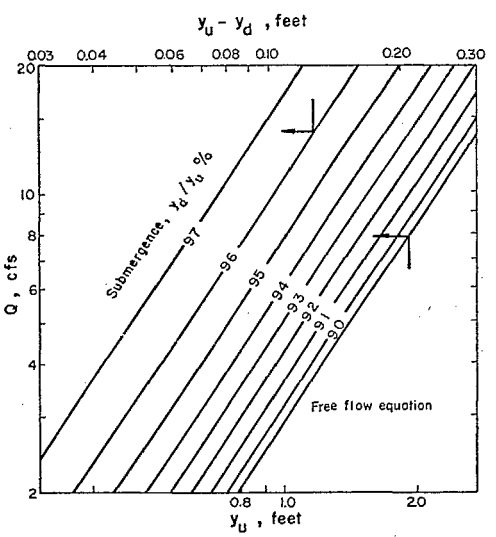
(a) Definition sketch of side constriction



(b) Example of relationship between π_2 and π_3



(c) Example of submerged flow calibration curves.



(d) Example of free flow and submerged flow calibrations

Figure 1. *Development of submerged flow calibration for side constriction*

Overflow constrictions

A number of methods for analysing submerged flow over weirs have been developed by various investigators. Typical of the approaches applied to this particular problem are those reported by Robinson [5] and Villemonte [9] as shown in Figures 2(b) and 2(c), respectively. In addition, the logarithmic submerged flow calibration shown in Figure 1 for side constrictions is equally applicable to overflow constrictions. All three methods of analysis have been compared by the authors [7] for data collected using various weir shapes. The comparisons showed each method of analysis to yield good results.

Using the definition sketch shown in Figure 2(a) [2], the general form of the approximate submerged flow equation is

$$Q = \frac{C_2 (h - t)^{n_1}}{[- (\log S + C_3)]^{n_2}}. \tag{3}$$

The general form of the free flow equation is

$$Q = C_1 h^{n_1}. \tag{4}$$

The value of the exponent n_1 is dependent upon the geometric shape of the overflow constriction. For example, n_1 is equal to 3/2 for a rectangular constriction and 5/2 for a triangular constriction. For a trapezoidal constriction, n_1 is equal to 3/2 for zero flow depth and 5/2 for an infinite flow depth.

Closed conduit constrictions

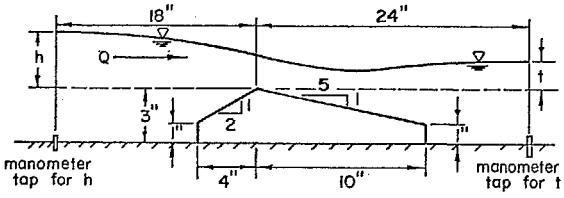
A number of closed conduit constrictions are utilized for measuring discharge. The most familiar device is the Venturi meter, which is used under pressurized (full pipe) flow conditions. The Palmer-Bowlus flume, which is shown in Figure 3(a), is frequently used as a critical-depth flume for measuring discharge in sewers. Although submerged flow ratings are not presently available for this flume, they could be easily obtained in the laboratory. A simple modified Venturi meter is shown in Figure 3(b). This meter could be rated for free flow, submerged flow, and pressurized flow. In fact, any type of closed conduit constriction could be rated for all three flow conditions.

FLOW RESISTANCE

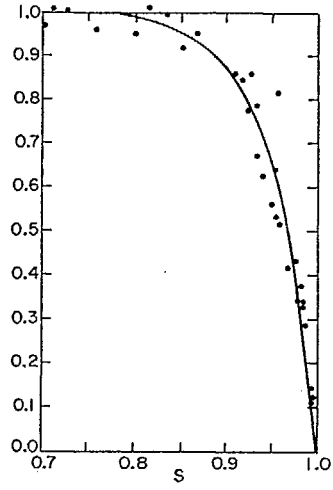
Overton and Hamon [4] have presented a technique for evaluating flow resistance. The technique is based on the principle that the shearing stress, τ, is proportional to V^2, where V is the mean velocity. For example, if the Chezy equation is used to describe the discharge in a rectangular channel

$$Q = CA\,(R\,S_e)^{1/2} \tag{5}$$

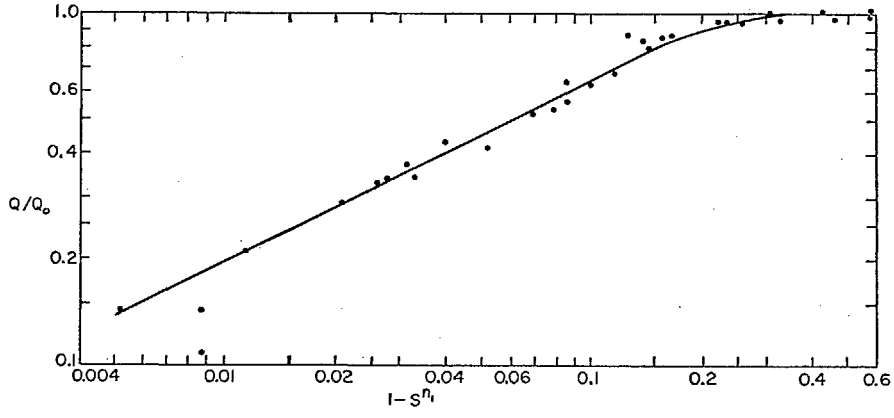
where C is the Chezy resistance coefficient, A is the cross-sectional area of flow, R is the hydraulic radius, and S_e is the slope of the channel bed, water surface,



(a) Test model Crump weir.



(b) Two-dimensional submerged flow plot of Crump weir data.



(c) Submerged flow plot of Crump weir data by Villemonte's technique.

Figure 2. *Submerged flow calibrations for overflow constriction*

and energy gradient. However, if the mean velocity is defined over the depth $y - y_s$, then

$$Q = CA' (R'S_e)^{1/2} \tag{6}$$

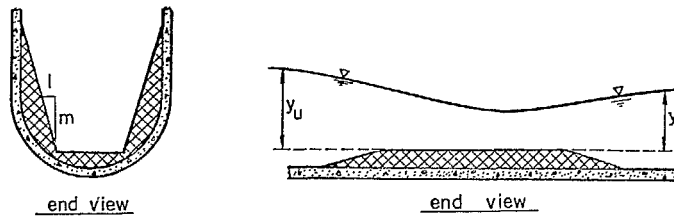
where

$$A' = w (y - y_s) \tag{7}$$

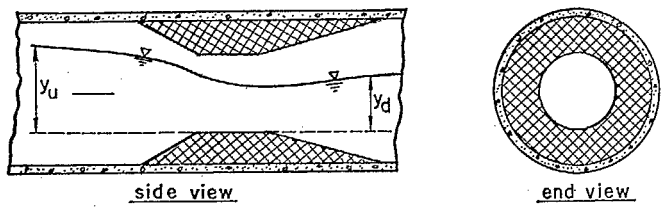
and

$$R' = \frac{w(y - y_s)}{w + 2(y - y_s)} \tag{8}$$

where w is the width of the channel.



(a) Palmer - Bowlus flume



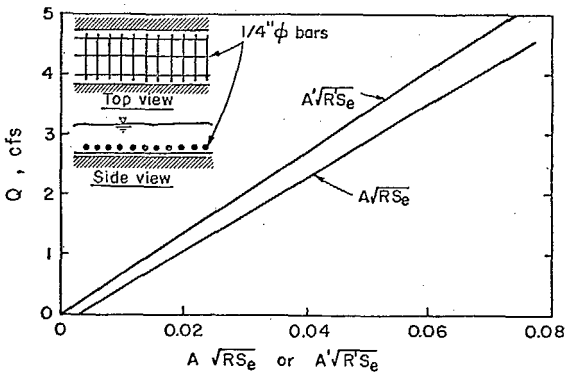
(b) Modified Venturi meter

Figure 3. *Free surface flow in closed conduit constrictions*

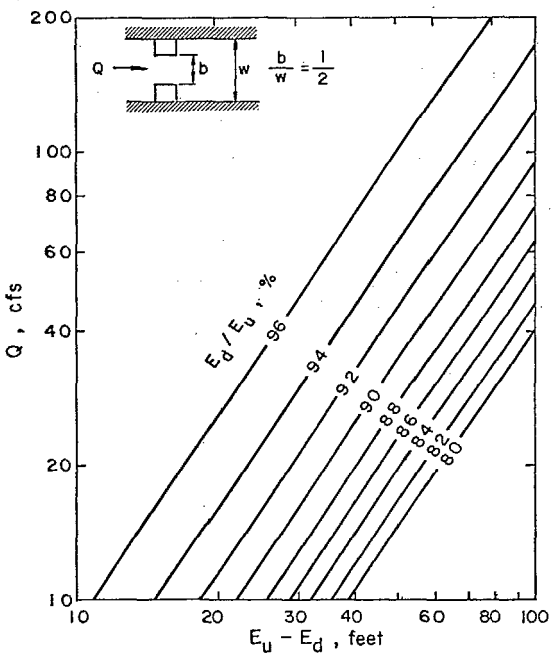
A trial and error solution is required to determine the magnitude of the roughness parameter, y_s. The value of y_s is obtained when a plot of Q against $A'(R'S_2)^{1/2}$ passes through the origin. The slope of the line passing through the origin yields the resistance coefficient. An example of such a plot is shown in Figure 4(a) [8]. The roughness parameter, y_s, will be referred to later in this paper in describing stage-discharge ratings.

BRIDGE CONSTRICTIONS

In order to relate the concept of a depth correction, or roughness parameter y_s, to the subcritical flow rating of bridge constrictions, experiments were conducted in the laboratory [8]. A flume 3 feet (90 cm) wide, 18 inches (45 cm) deep and 24 (3.70 m) feet in length was used. The slope of the flume could be varied. A roughness pattern was placed on the flume bed which consisted of 1/4-inch (6 mm) round rod spaced 6 inches (15 cm) longitudinally and 12 inches (30 cm) transversely. The total height of this roughness pattern was 0.041 (12 mm) feet. The plot for determining the Chezy resistance coefficient is shown in Figure 4(a), which was obtained by using a flow depth correction (roughness parameter y_s) of 0.03 feet (9 mm). Using an effective zero bed level of 0.03 feet (9 mm) above the flume bed, the free flow and submerged flow ratings for various bridge constrictions placed in the flume were determined.



(a) Determination of Chezy resistance coefficient



(b) Subcritical flow rating for simple vertical board bridge constriction

Figure 4. *Development of resistance coefficient and subcritical flow rating for simple vertical board bridge constriction*

The subcritical flow rating for a simple vertical board bridge model, in which half of the channel was constricted, is shown in Figure 4(b). Rather than using y_u and y_d in the rating, $E_u = y_u + V_u^2/2g$ and $E_d = y_d + V_d^2/2g$ were employed.

The importance of Figure 4(b) is that the abscissa is the true energy loss across the constriction, $E_u - E_d$. Thus, the plot has significance in portraying hydraulic efficiency or efficiency in energy dissipation for numerous types of open channel structures under subcritical flow conditions. Examples would be any type of energy dissipator or open channel expansions or transitions.

STAGE-DISCHARGE RATINGS

The stage-discharge relationship developed for a stream gauging station can be represented mathematically [1, 3] by the equation

$$Q = C_0 (y - y_0)^{n_0} \tag{9}$$

where y is the observed stage above an arbitrary datum and y_0 is a stage or depth correction. The stage correction, y_0, amounts to a correction in the datum level from which stage should be measured. A comparison of the corrected datum level with the characteristics of the channel bed, along with its roughness, is comparable to arriving at the roughness parameter, y_s. Since n_0 is a function of channel shape, studies regarding the effect of various channel shapes on n_0 would also assist in arriving at more accurate stage-discharge relationships as well as facilitating a more accurate determination of y_0 and consequently y_s. Thus, changes in the stage-discharge relation at a particular stream gauging station could be related to changes in channel shape and roughness. Unreasonable changes in either n_0 or y_0 could be detected and the rating suspected. A time history of changes in n_0 and y_0 could be developed and related to changes in channel shape and geometry, thereby providing a tool for evaluating the accuracy of historical run-off records at the same site. Although equation 9 is only valid for sites not affected by downstream conditions, this same type of thinking could be applied to stage-fall discharge ratings.

ACKNOWLEDGEMENTS

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BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Carter R. W., Davidian J., 1965. Discharge ratings at gaging stations. In : U.S. Geological Survey. *Surface Water Techniques, Hydraulic Measurements and Computations,* Book 1, Chapter 12.
2. Crump, E. S. 1952. A new method of gauging stream flow with little afflux by means of a submerged weir of triangular profile. *Proceedings, Institution of Civil Engineers London* , Paper No. 5848, p. 223-42. March.

3. Dickinson, W. T. 1967. *Accuracy of discharge determinations.* Hydrology Paper No. 20, Colorado State University, Fort Collins, Colorado, June.
4. Overton, D. E.; Hamon, W. R., 1970. Constant resistance coefficients and roughness parameters in rigid open channels. Paper presented at ASCE, Water Resources Engineering Conference, Memphis, Tennessee. 26-30 January.
5. Robinson, A. R. 1964. Water measurement in small irrigation channels using trapezoidal flumes. Paper No. 64-210, presented at 1964 annual meeting of ASAE, Fort Collins, Colorado. June.
6. Skogerboe, G.V.; Hyatt, M. L.; Eggleston, K. O., 1967. *Design and calibration of submerged open channel flow measurement structures:* Part 1, *Submerged flow.* Report WG 31-2, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah. February.
7. Skogerboe, G. V.; Hyatt, M. L.; and Austin L. H., 1967. *Design and calibration of submerged open channel flow measurement structures:* Part 4, *Weirs.* Report WG 31-5, Utah Water Research Laboratory, College of Engineering, Utah State University, Logan, Utah. May.
8. Skogerboe, G. V; Austin L. H; Kuan-Tao Chang. 1970. Subcritical flow at open channel structures: Bridge constrictions. Utah Water Research Laboratory, Utah State University, Logan, Utah. (Publication pending.)
9. Villemonte, J. R. 1949. Effect of submergence on discharge of sharp-crested weirs. Ph. D. dissertation, University of Wisconsin, Madison, Wisconsin.

DISCUSSION

Herschy: The author had included the Crump weir in his deliberations. E. S. Crump spent many years in India and received the Indian Government's highest civilian decoration. It was perhaps true to say that the Crump weir was born in India. There are now over 120 of these weirs in the United Kingdom ranging from very low flows to a capacity of 22,000 cusecs. The total cost of installation of these weirs is in the region of £1 million. Many of the weirs have a device incorporated in their design for submerged flow (or non-modular flow), and details of this are given in *Crump Weir Design,* Technical Note No. 3 (revised), 1970, Water Resources Board, Reading, United Kingdom.

Godoy: How can one relate the negative and positive backwater curve, and water levels, with the discharge in a construction? A negative backwater is present when a water level is drawn down, and a positive one when the water level is rising in a section.

Skogerboe: We have been concerned with very flat channels with slow velocities. The case mentioned by you with an unsteady flow has not been studied by me. I have worked only with steady flow conditions.

Flow over side-weirs

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Abstract. An experimental investigation of the flow over sharp-edged side-weirs in rectangular channels is reported in this paper. The parameters affecting the coefficient of discharge have been identified and an expression has been derived analytically for the variation of the discharge coefficient of a side-weir of zero height with the Froude number of upstream main channel flow. The experimental data show a very good agreement with this expression.

The discharge coefficient of side-weirs of finite height has been studied experimentally and is shown to be essentially the same for the corresponding side-weir of zero height.

ECOULEMENT SUR UN DEVERSOIR LATERAL

Résumé. Cet article présente une étude expérimentale d'écoulement sur barrage de cote de fil tranchant en canaux rectangulaires. Les paramètres affectant le coefficient d'écoulement sont identifiés et on en déduit une expression analytique pour vérifier les variations du coefficient d'écoulement d'un barrage de cote hauteur zéro en fonction du nombre de Froude de l'écoulement du canal principal à l'amont. Les résultats sont conformes à cette expression.

Les coefficients d'écoulement des barrages de cote de hauteur finie sont étudiés expérimentalement, et l'on montre qu'ils sont peu différents des valeurs obtenues pour le barrage de cote de hauteur zéro.

FLUJO SOBRE VERTEDEROS LATERALES

Resumen. En la presente comunicación, se expone una investigación experimental del flujo sobre vertederos laterales con borde afilado en cauces rectangulares. Los parámetros que afectan al coeficiente de caudal han sido identificados y se ha deducido una expresión analítica para la variación del coeficiente de caudal, de un vertedero lateral de altura cero, con el número de Froude del flujo del cauce principal aguas arriba. Los datos experimentales muestran estar muy en conformidad con dicha expresión.

El coeficiente de caudal de los vertederos laterales de altura finita ha sido estudiado experimentalmente y se ha visto que concuerda esencialmente con el correspondiente a vertedero lateral de altura cero.

ПОТОК ЧЕРЕЗ БОКОВЫЕ ВОКОСЛИВЫ

Резюме. Доклад посвящен экспериментальным исследованиям потока через тонкостенные боковые водосливы в прямоугольном русле. Были уточнены параметры, оказывающие влияние на коэффициент расхода, и получено аналитически математическое выражение для оценки изменения коэффициента расхода бокового водослива нулевой высоты с учетом числа Фруда основного руслового потока в верхнем бьефе. Экспериментальные данные показывают весьма хорошее соответствие с этим выражением.

Коэфициент расхода боковых водосливов ограниченной высоты был исследован экспериментально, причем оказалось, что он имеет в основном то же значение, что и для соответствующего бокового водослива нулевой высоты.

INTRODUCTION

A side-weir is defined as a free overflow weir set into the side of a channel with the purpose of allowing part of the liquid to spill over the side if the surface of flow in the channel rises above the weir crest. The applications of side-weirs can commonly be found in: (a) sanitary engineering practice, to separate storm flow in the combined sewer system; (b) irrigation engineering, to remove excess flow in a canal and as protection to flood embankments from overtopping.

The hydraulic behaviour of side-weirs has received considerable interest since the early twenties of this century. However, most of the early works were empirical in nature. Probably the first rational approach was made by De Marchi [ref. 3] in 1932. Even though there have been many other investigations on the problem of side-weirs, notably by Allen [2], Frazer [5] and Collinge [4], the effective contribution to knowledge is not significant. The present position in this subject is that no reliable method is available to predict the discharge over a side-weir or to predict the change in the water surface profile due to the resulting, spatially varied flow. This paper presents the results of an investigation on the flow over side-weirs (specifically the variation of De Marchi coefficient) undertaken at the Indian Institute of Technology, Kanpur, India.

DE MARCHI EQUATION FOR SIDE-WEIRS

The general differential equation of the spatially varied flow modified for a rectangular horizontal channel ($S_0 = 0$) of zero friction (i.e., $S_f = 0$) becomes [3]:

$$\frac{dy}{dx} = \frac{Qy\left(-\frac{dQ}{dx}\right)}{gB^2y^3 - Q^2} \tag{1}$$

where y = the depth of flow at any distance x from the beginning of the side-weir; B = width of the channel; Q = discharge flowing in the channel at a given section. (See definition sketch, Figure 1.)

The discharge over the side-weir per unit length (q_s) is assumed as

$$q_s = \frac{dQ_s}{dx} = -\frac{dQ}{dx} = \frac{2}{3} C_M \cdot \sqrt{2g} \cdot (y - s)^{1.5} \tag{2}$$

where s = height of the side-weir; C_M = a discharge coefficient, called here the De Marchi coefficient.

Since $S_f = 0$, E = specific energy, constant along the channel,

$$Q = By \cdot \sqrt{2g(E - y)}. \tag{3}$$

From equations 1, 2 and 3

$$\frac{dy}{dx} = \frac{4C_M}{3B} \cdot \frac{\sqrt{(E - y)(y - s)^3}}{3y - 2E}. \tag{4}$$

Integrating

$$x = \frac{3B}{2C_M} \cdot \phi\left(\frac{y}{E}\right) + \text{constant} \tag{5}$$

where:

$$\phi(y/E) = \frac{2E - 3S}{E - S} \cdot \sqrt{\frac{E - y}{y - S}} - 3 \sin^{-1} \cdot \sqrt{\frac{E - y}{E - S}} \tag{6}$$

ϕ = varied flow function of De Marchi.

For a side-weir of length L,

$$L = \frac{3}{2} \frac{B}{C_M} \cdot [\phi_2 - \phi_1] \tag{7}$$

where suffixes 1 and 2 refer to the beginning and end of the weir.

If conditions at section 1 are known (i.e. Q_1, y_1), ϕ_2 and hence Q_2 and y_2 can be found, provided C_M is known. The total discharge over the side-weir Q_s would then be

$$Q_s = Q_1 - Q_2 \tag{8}$$

DE MARCHI COEFFICIENT, C_M

Reliable information on the value of C_M corresponding to various flow conditions is lacking. Ackers [1] suggests a value of $C_M = 0.625$ if y is measured at a distance remote from the plane of the weir and $C_M = 0.725$ if y is measured at the plane of the weir. Apparently he assumes C_M to be constant. Collinge [4] finds that C_M varies with the mean velocity of the main channel.

Dimensional analysis indicates that

$$C_M = \text{fn}\left(F_1 = \frac{V_1}{\sqrt{gy_1}}, \frac{L}{B}, \frac{y_1}{L}, \frac{S}{y_1}\right). \tag{9}$$

It should be expected that F_1 would be a significant parameter affecting the value of C_M and the remaining parameters, representing the geometrical configurations of the flow, would have small effects, if any.

For a side-weir of zero height (i.e. $S = 0$)

$$C_M = \text{fn}\left(F_1, \frac{L}{B}, \frac{y_1}{L}\right). \tag{10}$$

Assuming L/B and y_1/L have very insignificant effects, an expression for the variation of C_M with F_1 for a weir of zero height is derived as follows.

VARIATION OF C_M WITH F_1 FOR A SIDE-WEIR OF ZERO HEIGHT

The flow over the side-weir can be considered as a deflected jet, as in Figure 1. Consider an elemental length dx of the weir. According to equation 2

$$Q_s' = q_s \cdot \mathrm{d}x = \frac{2}{3} C_M \sqrt{2g} \cdot \mathrm{d}x \cdot y^{3/2} \qquad (\because S = 0). \tag{11}$$

Since the effective width of the jet through this element is reduced such that $(\mathrm{d}x)_e = \mathrm{d}x \,.\, \sin\theta$, the discharge Q_s' can be rewritten as

$$Q_s' = \frac{2}{3} C_M^* \cdot \sqrt{2g} \cdot \mathrm{d}x \sin\theta \, y^{3/2} \tag{12}$$

where C_M^* is a constant coefficient. Also, as a first approximation in subcritical flow

$$\sin\theta = \sqrt{1 - \left(\frac{V_1}{V_j}\right)^2}. \tag{13}$$

Now it is assumed that the critical depth corresponding to q_s occurs at the plane of the side-weir of zero height such that

$$\frac{V_C^2}{2g} = \frac{1}{3} \cdot E \tag{14}$$

where

$$E = y_1 + \frac{V_1^2}{2g}. \tag{15}$$

Substituting $V_C = V_j$ and noting $F_1^2 = \dfrac{V_1^2}{gy_1}$,

$$\sin\theta = \sqrt{1 - \frac{3F_1^2}{F_1^2 + 2}}. \tag{16}$$

From equations 11, 12 and 16

$$C_M = C_M^* \cdot \sqrt{1 - \frac{3F_1^2}{F_1^2 + 2}}. \tag{17}$$

The value of C_M^* can be assumed to be 0.611 as it represents the outflow from a constriction with $F_1 \to 0$. With this

$$C_M = 0.611 \cdot \sqrt{1 - \frac{3F_1^2}{F_1^2 + 2}}. \tag{18}$$

Equation 18 could be expected to give the variation of C_M with F_1 for subcritical flow over a side-weir of zero height.

In supercritical flow, however, the discharge coefficient C_M could be assumed to be essentially independent of the Froude number as the zone of influence of the side-weir will be confined to its immediate neighbourhood only.

EXPERIMENTAL STUDY

An experimental investigation was undertaken at the Hydraulic Laboratories of the Indian Institute of Technology, Kanpur: (a) to verify equation 18 giving C_M for side-weirs of zero height; (b) to find the effect of other parameters on C_M; and (c) to find C_M for supercritical flow. The experiments were conducted in two horizontal channels:

Flume A: width 610 mm, length 9.0 m, bed: cement plaster.

Flume B: width 248 and 124 mm, length 3.0 m, bed: aluminium.

The ranges of variables studied are shown in Table I.

Table I. *Range of parameters studied*

Parameter	Range
F_1	0.02–4.3
L/B	0.2 –1.0
S/y_1	0.2 –0.96
y_1/L	0.1 –2.4

The weirs were sharp-edged and were fully aerated on the down-stream side. The depth y_1 was measured on the centreline at section 1, (i.e. $x = 0$). A total of 200 experiments were conducted and the results are shown in Figures 2, 3 and 4.

Subcritical flow

Figure 2 shows a plot of C_M calculated from experimental data for side-weirs of zero height, plotted against the Froude number of the flow. Only data for the subcritical region are shown in this plot. Also plotted are the variation of C_M given by equation 18. It can be seen that all the plotted points follow the variation given by equation 18 very well. It can be seen that there is no effect of the parameter L/B. A further examination revealed a similar insignificant effect of the parameter y_1/L. Thus it is concluded that the expression given by equation 18 correctly predicts the variation of C_M with Froude number for subcritical flow in side-weirs of zero height.

Figure 3 represents the plot of the experimental data on subcritical flow over side-weirs of finite height. A very good fit of the experimental data with the line given by equation 18 is indicated. It is interesting to note the absence of any significant effect of the parameter s/y_1 and also of the two parameters L/B and y_1/L. A slight observable deviation of data at a Froude number of 0.7 can be attributed to experimental errors.

Supercritical flow

Figure 4 shows a plot of C_M versus F_1 for supercritical flow over side-weirs of zero height. It is once again seen that there is no effect of the parameters L/B and y_1/L. Also, the effect of Froude number F_1 itself is very small, the variation being expressible as

$$C_M = 0.36–0.8F_1 \, . \tag{19}$$

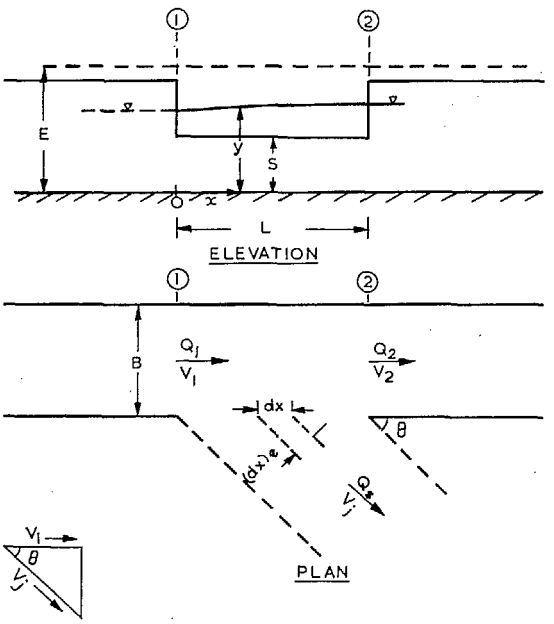


Figure 1. *Definition sketch*

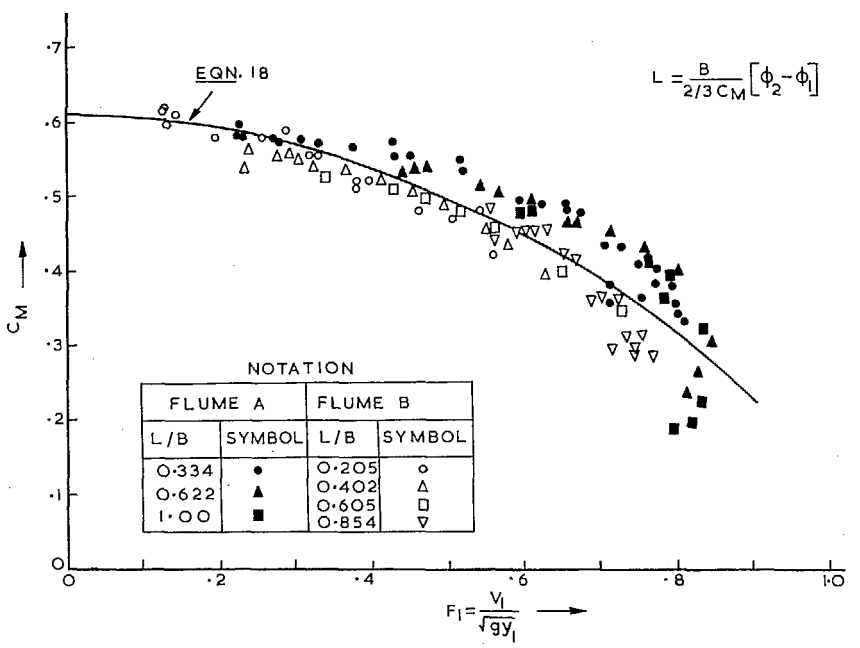


Figure 2. *Variation of C_M with F_1 for side weirs of zero height*

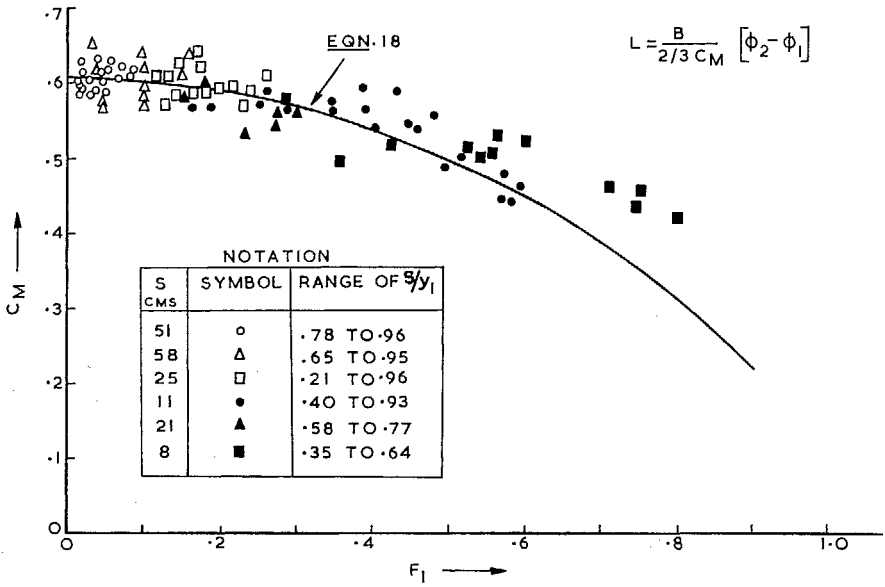


Figure 3. *Variation of C_M with F_1 for side-weirs of finite height*

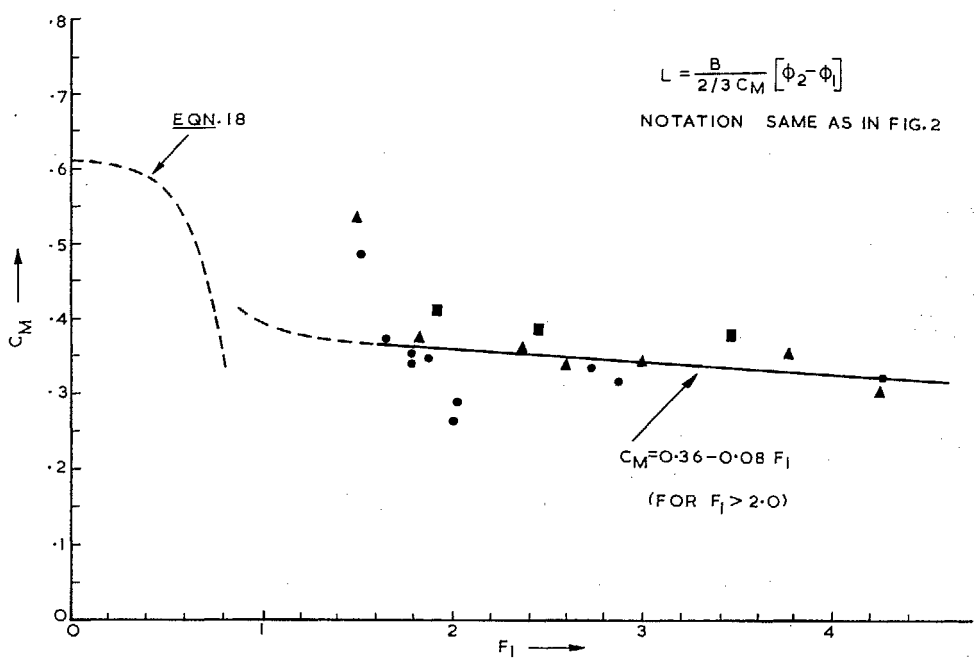


Figure 4. *Variation of C_M with F_1 in supercritical flow for side-weir of zero height*

It is possible that in an ideal case C_M would be independent of F_1. The small effect of F_1, seen in equation 19, is probably due to friction effects.

CONCLUSION

As a result of the investigation of the variation of the De Marchi discharge coefficient C_M, the following conclusions are drawn:

1. Equation 18 adequately represents the variation of C_M with F_1 in subcritical flow as confirmed in Figure 2.
2. The discharge coefficient C_M for side-weirs of finite height is essentially the same as that for side-weirs of zero height.
3. For supercritical flow, the discharge coefficient varies very slowly with the Froude number F_1, as indicated in Figure 4.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Ackers, P. A theoretical consideration of side-weirs as storm water out flows. *Proc. Inst. C.E. (London)*, vol. 6, February 1957.
2. Allen J. W. The discharge of water over side-weirs in circular pipes. *Proc. Inst. of C.E. (London)*, vol. 6, February 1957.
3. Chow, V. T., *Open-Channel Hydraulics*, New York, McGraw-Hill, 1959.
4. Collinge, V. K. The discharge capacity of side-weirs. *Proc. Inst. C.E. (London)*, vol. 6, February 1957.
5. Frazer W. The behaviour of side-weirs in primastic rectangular channels. *Proc. Inst. C.E. (London)*, vol. 6, February 1957.

DISCUSSION

Markovic: What is the physical reason for the same discharge coefficient at zero and finite height of the side-weir?

Subramanya: The physical explanation is most probably that the side-weir acts as a skimming weir in the sense that the velocity of the flow over the weir is predominant, unlike the case of an ordinary weir. In this case only that part of the flow which is above the side weir is important in deflecting the jet.

Gauging stations on sediment-loaded mountain rivers

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Abstract. Requests for gauging stations are increasing. Stations must now be built at unfavourable sites and on sections with sediment transport and, therefore, with changing bed profiles.

Moreover, there is a change in the character of the data required. It is no longer sufficient to adapt the curve representing the stage-discharge relationship after it had changed and in such a manner as to be able to indicate the discharge data only a considerable time after the events have occurred. Water management demands knowledge of instantaneous discharge data.

Artificial river sections for which the stage-discharge relationship is stable all the time, irrespective of sediment transport, had therefore to be created. The stations must be designed so that sediment is never deposited in the section which would influence the relationship.

A scheme for a gauging station fulfilling the above conditions is shown; the different elements forming the stations and their functions are explained; an example is drawn from an actual station.

STATIONS DE MESURE DU TRANSPORT DES SEDIMENTS DANS LES RIVIERES DE MONTAGNE

Résumé. Il existe un besoin croissant de stations hydrométriques ; il est souvent nécessaire de construire des stations sur des tronçons peu favorables de la rivière où des barrages naturels de rocher font défaut et où les alluvions charriées et partiellement déposées modifient souvent la forme du lit.

D'autre part, le type de données demandées varie. Il ne suffit plus d'adapter la courbe relation débit/hauteur limnimétrique après qu'elle a été modifiée, et d'indiquer ainsi les débits à posteriori avec un retard sur l'événement. L'aménagement des eaux exige de fournir des indications sur les débits au moment même où ces derniers se présentent.

Il était donc nécessaire de construire des tronçons artificiels de rivière sur lesquels la relation entre les débits et les hauteurs limnimétriques reste stable, malgré les sédiments transportés par la rivière. Les stations sont conçues de telle façon que les sédiments ne puissent en aucun cas se déposer dans le tronçon pouvant influencer ladite relation.

Un schéma représente une station répondant aux conditions mentionnées ci-dessus et des explications sont fournies sur les différentes parties de la station et leur fonctionnement. Un exemple est tiré d'une station actuelle.

ESTACIONES DE MEDIDA DE SEDIMENTOS EN RÍOS MONTAÑOSOS

Resumen. Están aumentando el número necesario de estaciones de aforo, y se tienen que construir estaciones en emplazamientos desfavorables y en secciones con transporte de sedimentos y por tanto con perfiles de cauce variable.

Por otra parte existe un cambio en el carácter de los datos que se precisan. Ya no es suficiente adaptar la curva que representa la relación nivel-caudal ; esta curva hay que modificarla periódicamente y por tanto se tienen los datos de caudal un considerable tiempo después de que han ocurrido los acontecimientos. Para la utilización de recursos se necesita que los datos de caudal se obtengan en el mismo momento en que se producen.

Por tanto es necesario crear secciones de río artificiales para las cuales la relación nivel-caudal sea estable durante todo el tiempo, con independencia del sedimento transportado por el río. Las estaciones se deben proyectar de manera que el sedimento no se deposite de ninguna forma en la sección ni que exista una influencia sobre la mencionada relación

Se presenta un esquema de una estación de aforo que reune las condiciones antes mencionadas ; se explican los diferentes elementos que forman la estación, las funciones de los mismos y se da un ejemplo de una de estas estaciones.

ГИДРОМЕТРИЧЕСКИЕ СТАНЦИИ НА ГОРНЫХ РЕКАХ С ОБИЛЬНЫМ ТРАНСПОРТОМ НАНОСОВ

Резюме. Требования к гидрометрическим станциямповышаются; станции должны создаваться в неблагориятных местах, где водосливы неприменимы, и на участках с интенсивным транспортом наносов и поэтому с меняющмися профилями дна.

С другой стороны, изменился и характер требуемых данных. Нельзя уже более считать достаточным пользоваться кривой расходов после того, как она изменилась и таким образом получать данные о расходах значительное время спустя после того, как события совершились.

Регулирование воды требует наличия данных о величине расходов в тот момент, когда они проходят.

Поэтому необходимо создавать искусственные участки, для которых кривая расходов устойчива в течение всего времени, несмотря на транспорт наносов, осуществляемый рекой. Эти станции должны бить рассчитаны таким образом, чтобы наносы никоим образом не отлагались на участке, в пределах которого имеется влияние на упомянутую зависимость.

Предложена схема гидрометрической станции, удовлетворяющая указанным требованиям; пояснены функции различных элементов, составляющих станцию. Показан пример действующей станции.

In a river with free flow the higher the water level, the larger the discharge. This is known from general experience. Formulae to be found in manuals on hydraulics have been established for geometrically defined channels and weirs and express the relationship between discharge and gauge height. For more complicated river beds, this relationship can be established by means of stream gauging measurements. Westernacher has shown this relationship in a very instructive manner for the special case of weirs with rounded tops, rectangular profile and horizontal river bed.

The relationship between discharge and gauge height is used for the operation of gauging stations in that the gauge height is recorded and the discharge determined from it. This is possible only in so far as the river bed retains its form, and this along the entire stretch which has an influence on the relationship mentioned. The cross-section of the river, for which the gauge height is recorded, has to be stable. Lifting or lowering of the ground at this section will, of course, change the relationship. Also the slope of the river bed has to be stable, because the current velocity depends on it. This is also important upstream of weirs, the velocity of approach and therefore the total energy head having to be taken into account. When the river bed changes, the relationship has to be re-established.

The condition of a stable river bed is not fulfilled in sediment-loaded rivers. After each flood, the form of the river bed is somewhat changed. Even where a weir stabilizes the cross-section, larger or smaller sediment deposits upstream of it will change the volocity of approach.

An example of this is shown in Figure 1. An existing weir was adapted to serve as a gauging station. The top of the weir was deepened in the middle part to concentrate the low water in a narrower bed, and thus to render the relationship between discharge and gauge height more precise. Records of the gauge height were made by a bubble gauge, the outlet of the compressed air being on the lateral wall on the left

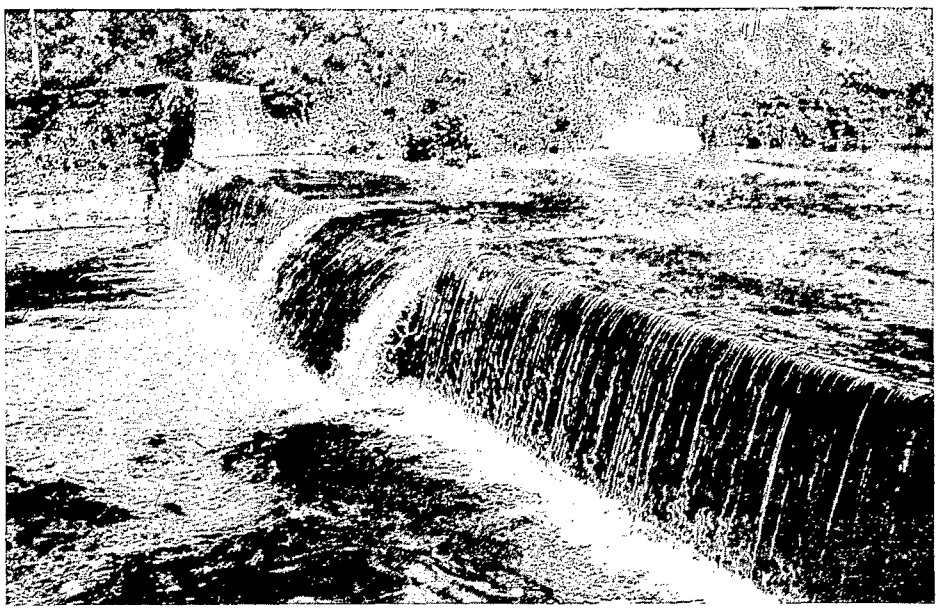


Figure 1. *Gauging station at Emmenmatt on the River Emme*

side of the low water channel. In spite of the weir top always being free from sediment, the relationship between discharge and gauge height changed as is shown in Figure 2.

Such changes complicate the work. Thus, the idea was conceived of prolonging the low water channel in an upstream direction with a slope that would exclude sediment deposits in order to get a stable relationship. Further development of this idea then led to the proposal for a system of constructions in the river bed. At first, the implementation of the suggested system was delayed because of the substantial cost but action was finally taken for the reasons set out below.

The needs for discharge data for design purposes increased as claims upon water for all kinds of purposes intensified. New gauging stations had to be built, sometimes on sites unfavourable from a hydraulics and sediment transport point of view, and where the construction of a station would, in the past, have been considered impossible. In addition, there was a change in the way stations were used.

In the past, it sufficed to establish the discharge by determining the relationship for different periods on the basis of the results of stream gauging measurement.

Today it is necessary to know the actual discharge at the moment when it occurs. The exploitation of hydraulic power plants and the regulation of lakes depends upon knowledge of the prevailing hydrological situation. For this purpose, instruments for the teletransmission of gauge height were installed. In order to determine discharge from the stage measurement the gauging station must be so constructed that the respective relationship does not change. These reasons were sufficient and the money was made available.

The fundamental idea was that all the water, including flood flow, has to flow through an artificial channel. At an appropriate point on the channel, the gauge height is recorded. For this point, the relationship between discharge and gauge height has to be stable. This means that the cross-section corresponding to this point

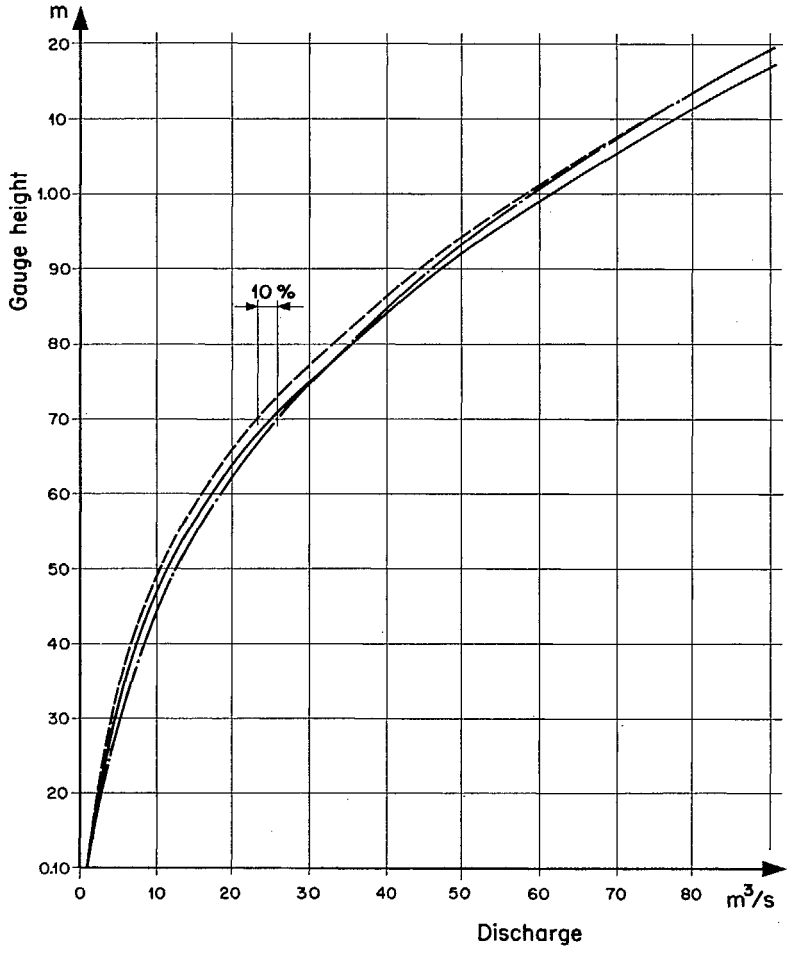


Figure 2. *Stage-discharge relationship for the station shown in Figure 1*

must be free of sediment deposits (first condition) and the slope of the water surface, corresponding to a particular discharge must at all times be the same (second condition).

The slope of the channel is such that the water flows with supercritical velocity. Sediment is not deposited at such velocities and therefore the first condition is fulfilled. The second condition would be fulfilled in uniform flow, that is with the same slope for channel, water surface and energy line. It is known that uniform flow exists only in rather long channels and in most cases it is not possible to build such long channels. The second condition can nevertheless be fulfilled without having uniform flow. It is only necessary that, for a given discharge, the water enter the channel each time at the same velocity as that at which it leaves the channel.

Figure 3 shows how this problem was solved. Upstream of the upper end of the channel there is a weir so arranged that a scour hole is produced between this weir and the entrance of the channel. It is important that the scour hole touch the channel. The water flows from upstream towards this weir, sometimes at higher,

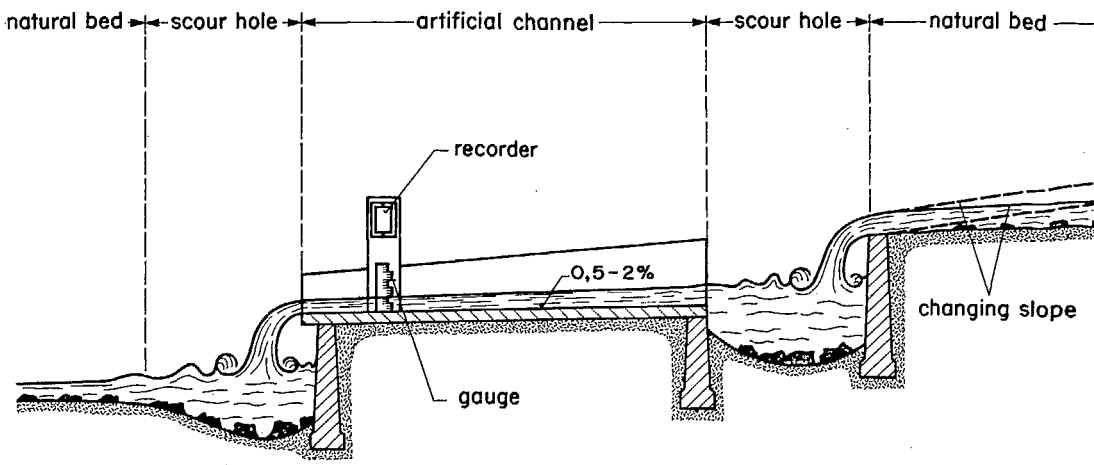


Figure 3. *Schematic longitudinal section of a gauging station on a sediment-loaded mountain river*

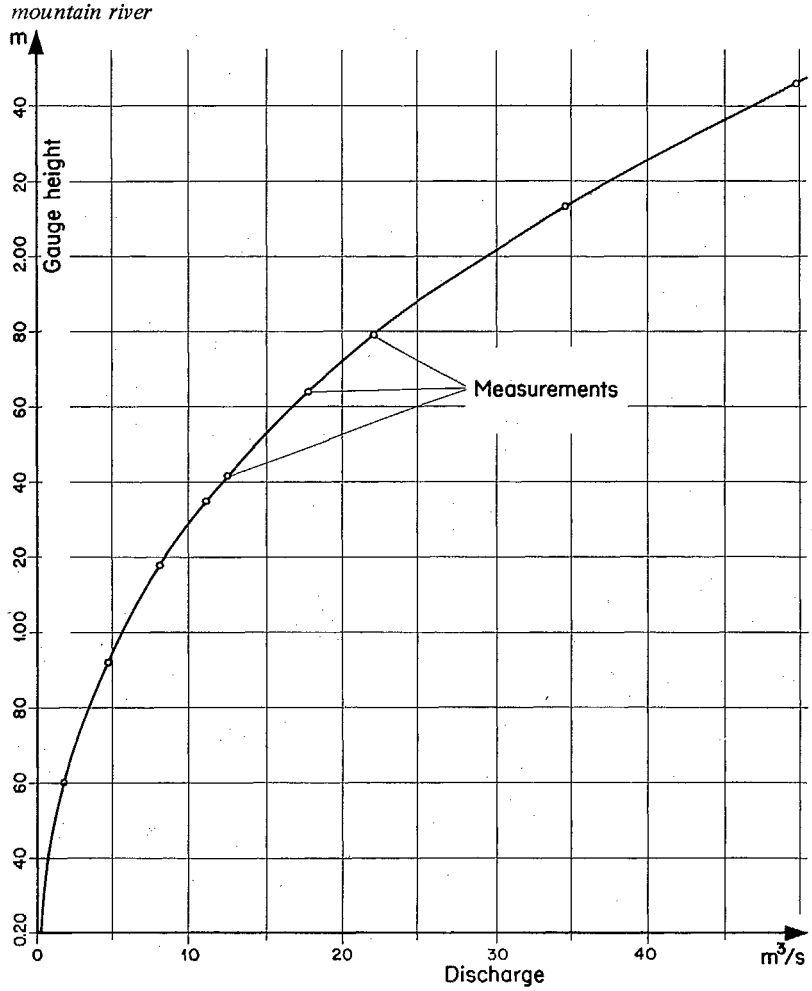


Figure 4. *Stage-discharge relationship for the station shown in Figure 5*

sometimes at lower velocity, depending on the state of the bed. But in the turbulent conditions of the scour hole, which is broader than the channel, this is equalized. The entrance into the channel is not influenced by the changing approach velocities to the weir; it is only conditioned by the height of the water surface in the scour hole. This height depends on the discharge entering the channel, which, under steady conditions, is equal to the discharge arriving from upstream.

Strictly speaking, the distance between the upstream weir and the entrance into the channel should correspond to the maximum possible flood. This distance would then be rather long and the danger exists that the scour hole could be partly filled up when the floods decrease. This must not happen. This distance must therefore be optimised, so that the station will function as described for small as well as for average discharges. In consequence floods will overflow the scour hole. This can be tolerated because of the somewhat lesser need for accuracy in flood data.

The water leaves the channel forming a free nappe. It has been described how the entrance conditions into the channel are always the same for a certain discharge. Thus, we have the same conditions of flow all along the channel, even when the flow is not uniform. For this reason, the overflow at the downstream end of the channel is always subject to the same conditions in the case of a given discharge.

The cross-section in which the gauge height is recorded is situated upstream of the depression range of the downstream overflow. In a cross-section corresponding to the former condition, a certain change in discharge causes a bigger difference in gauge height than at the end of the channel. The relationship between discharge and gauge height is therefore more accurate. On the other hand, the greatest possible distance from the entrance is required, because the water surface gets smoother in the downstream direction.

Twelve stations in the Swiss network have been constructed on the basis of these ideas. They are working well, thus confirming that a stable relationship can be obtained in this way (Fig. 4).

The fundamental ideas for the scheme were derived from the experience acquired with previously constructed stations. Such experimental stations are expensive, and their equipment should therefore lend itself to change in the parameters. For this reason, the need for model tests arose, and in 1958 the hydraulic laboratory of the Federal Polytechnic in Zürich incorporated this problem into its research programme, thus opening the way to a scientific analysis and completion of the problem. This happened at just the right moment, because a gauging station had soon to be built which would be exposed to extremely difficult conditions, i.e. the gauging station on the River Massa downstream of the Aletsch Glacier. For the design of this station, special model tests were made.

The station is shown in Figure 5. Its main data are as follows:

Largest measurable discharge	150 m^3/s
Distances along the channel axis:	
– from the upstream weir to the beginning of the curved entrance walls:	7.20 m
– from the beginning of the curved entrance walls to the beginning of the prismatic channel:	9.00 m
– length of the prismatic channel:	12.50 m
– distance between the topping point and the downstream end of the channel:	3.30 m
Maximum width of the scour hole downstream of the upper weir:	14.50 m

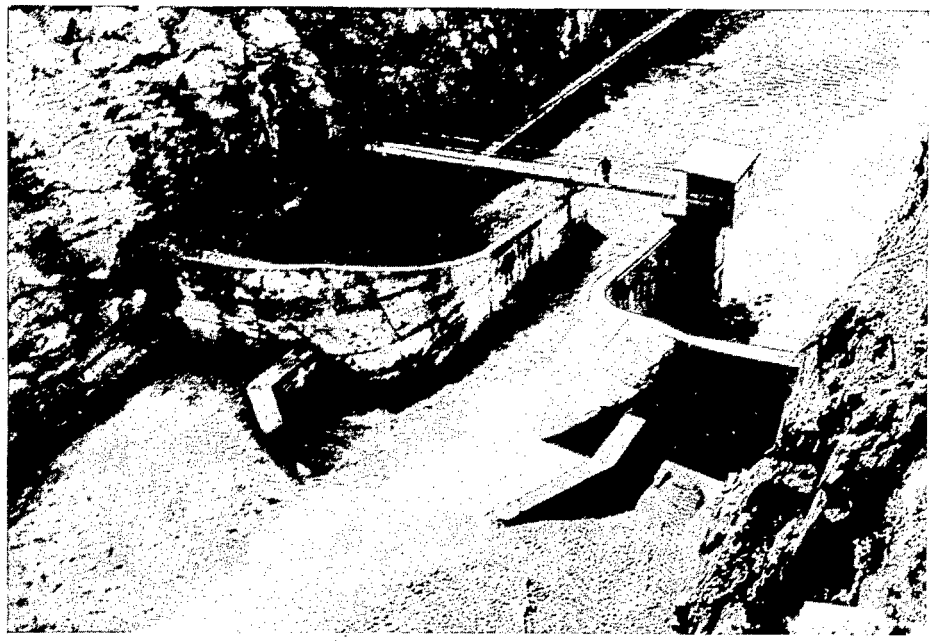


Figure 5. *Gauging station on the River Massa downstream of the Aletsch Glacier*

Width of the channel:	
– at the bottom	0.90 m
– between the vertical walls	6.00 m
Total depth of the channel:	
– at the beginning of the prismatic section	5.30 m
– at the downstream end	4.10 m
Slope of the channel bottom	5 ‰

The gauging station on the River Massa has to serve two power plants, situated downstream, which share the water according to a rather complicated programme. The intake is controlled automatically, one of the controlling factors being the discharge of the river which is teletransmitted from the gauging station.

The discharge statistics of the River Massa are also an important element for a general knowledge of the country's water resources. For about a hundred years, glaciological investigations have been made on the Aletsch Glacier. In this connexion, too, the knowledge of the discharge of the River Massa is necessary and important.

The cost of the construction of the Massa station was about Sw.Fr. 850,000. Because of its importance for basic scientific research, the Swiss Federal Authorities decided to contribute one-third of the cost, the rest being paid by the two companies concerned.

The station went into operation in spring 1965 at the beginning of the International Hydrological Decade, and its construction can be considered as a contribution to this important scientific project.

DISCUSSION

Burz: 1. Is the bedload deposited in the stilling basin or is it transported in the measuring channel?

2. In mountain streams a determination of the discharge with an accuracy of ± 10 per cent is normally sufficient. Therefore do you not agree that a measuring construction with a cascade is acceptable?

3. Do you have any experience with retarding basins for bedload for possible use for the measurement of discharges?

Walser: The only purpose of the cascade is to isolate the measuring channel from those parts in which sedimentation may occur. After passing the upper cascade, the bedload falls into the stilling basin and will be transported by a flood. Therefore all bedload will pass the measuring channel and will then either sediment or be transported further. The walls of the channel need special attention since they are subjected to great stress. The inlet is stressed by blows, the walls themselves by friction. This required different materials. The walls have been paved with an artificial stone which is very resistant to friction but too brittle to withstand blows. Here we have successfully applied wood, which is much more successful than steel or granite.

In my paper I have shown a station where we could apply only one swell which caused some uncertainties. We supposed therefore that a construction with one swell also in mountain streams would cause major uncertainties. Unfortunately I have no data at hand to demonstrate this.

I am sure that the provision of one swell is already an improvement on the natural state. It is difficult to judge the degree of this improvement since this depends on the local particularities. Near Davos we have built a station consisting of two consecutive swells without a channel in between. This river however has very little sediment transport and a very balanced régime, without sudden and immense floods. In this case this small construction was sufficient, but this is an exception in Switzerland.

Burz: In the Alps, many retarding basins for bedload have been built with a rectangular or trapezoid overfall. Did you make any investigations for utilizing these overfalls for the measurement of discharge?

Walser: There are only a few retarding basins. Generally they have been built in very small rivers in which discharge measurements have not yet been carried out. I am sorry that I cannot report on my experiences.

Kinori: 1. Mr. Walser said that the slope of the flume is determined in a way so that the flow will *always* be supercritical. How is this criterium achieved?

2. If the flow in the flume becomes subcritical there will be a hydraulic jump, downstream of the scourhole.

3. If the flow is slightly supercritical there will be waves.

Walser: The slope was chosen so as to achieve supercritical flow at least at major discharges. You are right—and I did not mention this—that the velocity may sometimes fall too low for supercritical flow, but in these cases no bedload is being transported. It is therefore only necessary for the flow to be supercritical when bedload transport commences in the river.

A hydraulic jump does not occur since the velocity increases towards the lower overfall. The channel ends with an overfall permitting free downfall of the water. The acceleration of the water is welcomed because of the fact that each particle entering the channel will leave it.

In our first construction we observed the well-known diagonal waves which may occur in channels with supercritical flow. They can be excluded by a suitable construction of the inlet structure. A trumpet form has been found to be the most suitable.

Mukeli: Comment changent les dimensions géométriques des tronçons artificiels suivant la grandeur des rivières ?

Walser: The dimensions depend on the size of the river. We have found the best dimensions with the help of models and also have evaluated practical experiences. There are also theoretical considerations for finding the dimensions.

Flow measurement of low-gradient streams in sandy soils

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Abstract. Few satisfactory natural controls are found in the low-gradient streams of the Southern Coastal Plain, United States, and accurate discharge data are scarce. Artificial control difficulties involve backwater effects and lack of free overfall, dictating a compromise between weir submergence and ponding. Structure stability and scour protection cause special difficulties in the noncohesive soils of the region.

Design, construction, and maintenance details are described for submerged broad-crested V-notch weirs used for flow measurements in Little River Experimental Watershed near Tifton, Georgia, Weir rating techniques, including model studies and volumetric, pitot tube, and current-meter methods for the prototypes, are presented. Methods are given for simultaneously recording upstream and downstream stages with digital-type recorders and subsequent development of rating equations for computing flow data which are used in watershed hydrologic analyses.

MESURE DE L'ECOULEMENT DES COURS D'EAU DE FAIBLE PENTE DANS LES SOLS SABLONNEUX

Résumé. On trouve peu de contrôles naturels satisfaisants dans les cours d'eau de faible pente de la plaine de la côte sud des Etats-Unis (Southern Coastal Plain, Etats-Unis d'Amérique), et on peut difficilement trouver des éléments d'information précis sur le débit. Les difficultés de contrôle artificiel proviennent des effets de remous et d'un manque de déversement libre, nécessitant un compromis entre la submersion du déversoir et la formation d'une nappe d'eau. La stabilité de construction et la protection contre l'affouillement causent des difficultés particulières dans les sols non cohérents de la région.

L'article fournit des détails sur les déversoirs triangulaires noyés à seuil épais, utilisés pour les mesures d'écoulement dans le bassin hydrographique du fleuve Little (Little River Experimental Watershed) près de Tifton (Georgie) : dessin, construction, entretien. Les techniques d'étalonnage des déversoirs, avec études sur modèles, méthodes volumétriques et méthodes utilisant le tube de Pitot et le moulinet font également l'objet d'une description. Des méthodes sont données pour l'enregistrement simultané dans les sections amont et aval, à l'aide d'enregistreurs numériques et de techniques de calcul des équations de débits utilisées pour obtenir les données nécessaires aux analyses hydrologiques d'un bassin.

MEDICIÓN DE CAUDAL EN CURSOS DE AGUA DE BAJO GRADIENTE EN SUELOS ARENOSOS

Resumen. Pocos controles naturales satisfactorios se encuentran en las corrientes de bajo gradiente en la llanura litoral del sudeste en Estados Unidos de América, y son escasos los datos precisos de caudal.

Las dificultades de un control artificial incluyen los efectos de remanso y la falta de pendiente imponen una solución entre vertedero sumergido y embalse. La establididad de estructura y la protección de recorrido originan dificultades especiales en los suelos no cohesivos de la región.

Se describen detalles de construcción, proyecto y mantenimiento para los vertederos de cresta amplia con entalla en V para las mediciones de caudal en la Pequeña Cuenca Fluvial Experimental, cerca de Tifton, Georgia. Las técnicas de calibrado de vertedero, incluyendo estudios de modelo y volumétricos, tubo de pitot y métodos de medición de corriente para los prototipos, figuran en el mismo. Se dan métodos para el registro simultáneo de niveles aguas arriba y aguas abajo con registradores de tipo digital y el desarrollo de las ecuaciones de calibración para el cálculo de datos de caudal que se utilizan en estudios de cuencas hidrográficas.

ИЗМЕРЕНИЕ РАСХОДОВ В ПОТОКАХ С МАЛЫМИ УКЛОНАМИ В ПЕСЧАНЫХ ГРУНТАХ

Резюме. Мало удовлетворительных естественных контролей имеется в потоках с небольшими уклонами на равнинной части южного побережья США, и точные данные о расходах редки. Устройство искусственных контролей затрудняется влияниями нижнего бьефа и отсутствием свободного перепада, вынуждая к принятию компромиссного решения между затопленным водосливом и водоёмом. Устойчивость сооружения и предупреждение размыва вызывают особые трудности в несвязных грунтах этого района.

Описываются детали расчёта, конструкции и эксплуатации для затопленного водослива с широким порогом, использованного для измерений стока на экспреиментальном водосборе близ Тифтона, штат Джорджия. Методика тарирования водослива, включая модельные опыты и объёмный способ, трубки Пито, а также вертушечные способы для натуры, тоже изложены. Предложены методы одновременной записи самописцами (цифрового типа) уровней вернего и нижнего бьефа и последующего получения расчетных уравнений для подсчета данных о стоке, используемых в гидрологическом анализе.

INTRODUCTION

Among the top priority needs of such action agencies as the USDA Soil Conservation Service are run-off studies on agricultural watersheds within the Coastal Plain and adjacent Flatwoods Land Resource Areas of the south-eastern United States. There is a scarcity of precise data on and a lack of satisfactory procedures for predicting run-off and water yield from these watersheds [refs. 1, 2].

These data are urgently needed to provide a basis for the planning of water supply for municipal use, irrigation, and pollution abatement. They are also required for the design of flood control and retardance structures used in a comprehensive watershed protection and flood prevention programme.

Research is needed to establish improved understanding of the factors affecting rainfall-run-off relationships in this area and to develop procedures for accurately predicting run-off rates—both instantaneous and total-flow volumes—from ungauged watersheds.

To meet these needs, studies are under way by the South-East Watershed Research Center, SWC-ARS. As a part of these studies, precise and extensive measurements of stream discharge are being made on various-sized channels within the 145-square mile (376 km^2) Little River Watershed near Tifton, Georgia.

THE PROBLEM

The Coastal Plain region presents a unique and difficult problem in the measurement of discharge from the low-gradient streams of the area.

Watersheds in this region are characteristically marked by broad, flat, alluvial flood plains with low-gradient, poorly defined channels. Above the lowest flows, channels become non-existent, with flows spreading across the wide wooded flood plains. For reasons of economy, flow-measuring installations must be placed at highway bridges or culverts—the only points where flow can be properly confined with any degree of practicality.

Streambed elevations on the Little River Watershed drop only 3 to 5 feet per mile (about 0.6 to 0.9 metre per kilometre)—a slope of less than 0.1 per cent.

Two problems are posed by the low-gradient streams. Free-outfall as required by most flow-measuring devices is difficult to obtain. Raising the control as little as 2 feet (0.6 metre) may cause ponding as far as one-half mile (800 metres) upstream. Therefore, a compromise between backwater ponding effects and submergence of the control must be made when designing and constructing the control.

THE APPROACH

The broad-crested Virginia V-notch weir has been found to be an economical and efficient flow measuring installation when properly designed and located with respect to highway culverts [3]. This weir has the advantages of sensitivity at very low flows, accuracy over a wide range of flows, and relative economy of installation and maintenance. Its greatest disadvantage is that flow conditions are generally rated for free outfall. However, in the low-gradient streams of the Coastal Plain, free discharge may be obtained only at the lower flow rates. Since weir crests must be installed relatively close to channel beds to reduce backwater ponding effects, weirs of a permissible height may be subject to submerged flow during much of the discharge period. Model studies indicate that submergence effects begin when the downstream stage inundates about 60 per cent of the upstream head. Free fall with nappe aeration is prevented, and flow depth at the weir crest becomes greater than critical depth.

Under submerged flow conditions, a change in downstream flow depth causes a change in upstream flow depth for a given constant discharge rate. Weir ratings based entirely on upstream levels may no longer be applicable. Both upstream and downstream heads are necessary. Therefore, streamflow measuring installations in Little River watershed are designed to record both headwater and tailwater elevations simultaneoulsy.

Design of measuring structures. Measuring facilities are designed so that all flow is contained within the V-notch portions of the weir during 90 to 95 per cent of the time. This is approximately two-thirds of the total flow volume, based on anticipated 25-year flow rates. While the measuring weir is designed for accurate measurements ranging from zero discharge to the 25-year flood, ample freeboard is allowed to pass the 100-year flood without damage to the structure.

Weirs are placed upstream from box culverts and downstream from bridges. Each is constructed on top of an interlocking steel-sheet piling cut-off and support wall. The piling is driven through the loose unconsolidated alluvial material into the undisturbed parent material below. Depth of this alluvium, as determined by subsurface borings, ranges from about 7 feet (2.1 m) in the upper end of the watershed to about 20 feet (6.1 m) at the downstream discharge site.

Weir designs are the same at both culverts and bridges—only guide wall arrangements and weir locations differ. Guide walls serve as sides of a large flume to force all flow over the measuring section. They are installed to a sufficient height to safely contain the anticipated 25-year flow, with an additional 3 feet (0.92 m) allowed for safety to carry the probable 100-year flood without overtopping losses.

At highway bridges these guide walls are constructed of the same type of interlocking steel-sheet piling and driven to approximately the same depth as the cut-off wall beneath the weir. Each guide wall extends about 50 feet (15.2 m)

downstream from the bridge abutment. At the bridge abutment, guide walls are turned out parallel to roadways for a distance of 8-10 feet (about 3 m). These wingwalls, driven into road shoulders, are necessary to prevent flow from passing outside guide walls and eroding the steep roadbanks.

The weir is placed about 25 feet (7.6 m) downstream from the bridge.

At box culverts, existing concrete wingwalls as built by highway departments serve as guide wall-wingwall combinations. However, tops of the sloping wingwalls are raised to a horizontal elevation about 1 foot (0.3 m) above the culvert ceiling. This modification prohibits flow spilling over wingwalls and bypassing the measuring section. Weirs extend the full length between the outer ends of the upstream culvert wingwalls.

A concrete water dispersal apron for erosion control is required immediately downstream from each weir. This 12-inch (0.3 m) thick slab which extends some 25 feet downstream (7.6 m), is supported by 8-inch-diameter (20 cm) treated timber piles driven to refusal. At highway bridges, rip-rap is placed for an additional 25 feet (7.6 m) downstream of the apron. This is not generally required below culverts.

Both upstream and downstream water surface elevations must be recorded to rate structures precisely during submerged conditions. This necessitates the use of dual stilling wells at each site. On top of the 8-inch-thick (20 cm) concrete walls of stilling wells, a recorder shelter is constructed monolithically with the wells.

Two Fischer-Porter digital-type stage recorders[1] are utilized to record water surface elevations. These recorders are connected to a common electronic timer with interfacing slave-drive in each recorder shelter. This arrangement provides a simultaneous punchout of both upstream and downstream stages. All timers throughout the watershed are the precise quartz-crystal oscillating type and are synchronized so as to give practically simultaneous readings at all sites—streamstage recorders, groundwater recorders, and raingauges. This arrangement produces a synoptic record of existing hydrologic conditions for the entire watershed at 5-minute intervals.

RATING OF STRUCTURES

Rating has been by model testing, complemented by field measurements.

Model tests. Model studies were conducted for the purpose of determining optimum design specifications as well as for rating control structures. Large weirs at bridge sites were modelled by Ree and Gwinn at the ARS Hydraulics Laboratory, Stillwater, Oklahoma.[2] Sheridan conducted model studies at the University of Georgia for smaller weirs at culvert sites.[3] Field discharge measurements are being made to correlate ratings derived through these model tests.

Design results. Gwinn and Ree constructed and tested models of sites A and B with drainage areas of 145 and 126 square miles (376 and 326 km^2) respectively, to a scale ratio of 1 : 30.

1 Brand names used in this paper are for identification purposes only, and do not imply an endorsement by the United States Department of Agriculture.

2 Personal correspondence from W. O. Ree and W. R. Gwinn.

3 Sheridan, Joseph M. Discharge Characteristics of Submerged Broad-Crested V-notch Weirs. Graduate thesis, Agricultural Engineering Department, University of Georgia, Athens, Georgia, 1968.

Site A consists of three concrete girder bridges on U.S. Highway 82 west of Tifton, Georgia. Model criteria provided for means of measuring prototype discharges from zero to 20,000 cfs (566 m^3/sec) with a maximum range of heads of 12.2 ft. (3.7 m). A number of tests were made with various flow rates. However, at each flow rate discharge was held constant, while a series of tests were conducted with varying tailwater elevations. Both headwater and tailwater elevations were measured using piezometers and recorded.

Test results indicate no change is needed in basic weir design from that discussed earlier. However, a change in guide walls and apron is found to be desirable. Guide walls extend at right angles from bridges to a point just below weir crest. At this point, walls are turned outward 45°, and extend 10 feet (3 m) farther downstream. The concrete apron extends below the weir for this 10 feet (3 m) rather than 25 feet (7.6 m) as discussed earlier. Energy dissipation is provided by two rows of 9 x 9 x 9-inch (22.86 cm) baffle blocks below the V-notch section and end sills at the downstream end of the apron as shown in Figure 1. Rip-rap is not found to be needed in this design.

Rating of large weirs: Weirs were rated for both prototype free flow and submerged conditions. Free flow discharge was found to conform to the equation

$$Q = C \left[\Sigma L_i H_i^{1.5} + \tan \frac{\theta}{2} h_v^{2.5} \right] \quad (1)$$

where: Q = discharge, cfs
C = discharge coefficient, $ft^{0.5}/sec$
L = crest length of flat crest, ft
H = head above flat crest, ft
θ = total angle of V-notch weir, degrees
h_v = head above V-notch, ft
i = subscript for different levels of flat crest not included in the V-notch section.

Submergence tests were run using a constant discharge rate and varying the tailwater elevation, while recording headwater and tailwater stages. Submergence ratios are computed by dividing tailwater head by headwater head with zero head being the level of the low point, or control. Discharge rates begin to be reduced significantly when submergence ratios reach approximately 60-65 per cent.

The discharge-submergence relationship obtained from these model tests is

$$Q = Q_{free} \left[1 - C(e^{T/H})^D \right] \quad (2)$$

where: Q = actual discharge, cfs
Q_{free} = discharge, cfs, from the free-flowing equation
C and D = constants
e = 2.718 . . . .
T = tailwater elevation minus zero elevation
H = headwater elevation minus zero elevation.

Rating of small weirs. Sheridan constructed geometrically similar models of broad-crested V-notch weirs at a typical box culvert to a scale ratio of 1 : 10. Scale models of prototypes with side slopes of 5 to 1 as well as 10 to 1 were constructed and

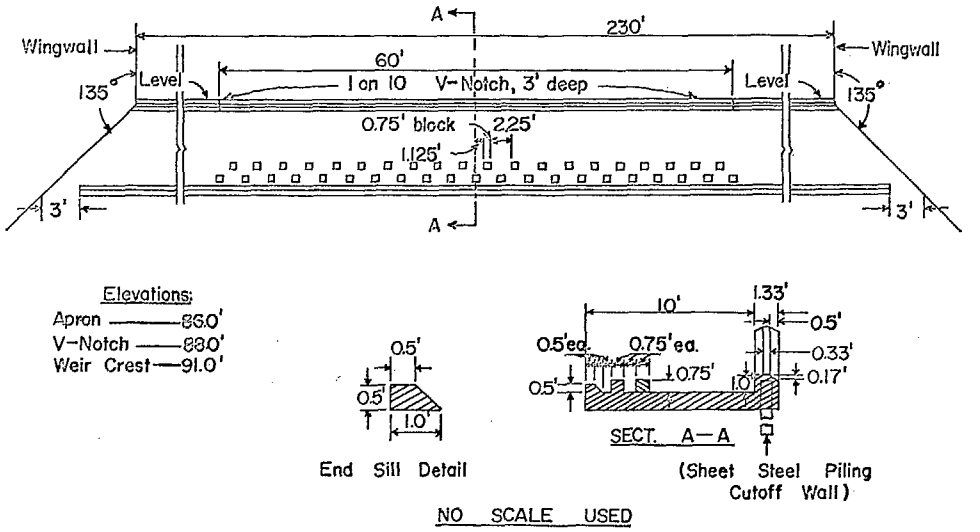


Figure 1. *West bridge weir and apron. Station B, Little River Watershed near Tifton, Georgia*

studied. Test procedures were essentially the same as those used in large weir tests, except that all flow was confined to the V-notch.

Results obtained for 5 to 1 and 10 to 1 V-notches combined into one rating equation (equation 3) compare favourably with those obtained by Ree and Gwinn (equation 4) [4] and Harrold and Krimgold (equation 5) [5].

$$Q = 2.50 \tan \frac{\theta}{2} H^{2.71} \tag{3}$$

$$Q = 2.90 \tan \frac{\theta}{2} H^{2.94}, \qquad H < 1.00 \text{ ft} \tag{4}$$

$$Q = 2.65 \tan \frac{\theta}{2} H^{2.58}, \qquad H > 1.00 \text{ ft}. \tag{5}$$

Field measurements. Weirs are rated using discharge measurements obtained through volumetric, pitot tube - pressure differential measurement, and current meter techniques. Ratings developed are used in verification of model tests results. Surface tension effects cause hydraulic model calibrations to be unsatisfactory up to model flow depths of about 0.04 foot (1.22 cm). Using a model scale of 30 to 1, no model calibration tests are obtained for prototype depths less than 1.2 feet (0.366 m).

Volumetric measurements. Very low flows are merely caught in a pan of known volume. Time required to fill the pan is measured with a stopwatch to determine flow rates.

Pitot tube - pressure differential measurements. A pitot tube connected to a differential pressure transducer and battery-powered dial indicator is mounted on a dolly

which traverses the length of the weir. Point measurements are made by an operator inserting the pitot tube by means of a vertical adjustment mechanism into the flow path at the proper depth—generally at 0.6 depth. Differential pressures are then measured on the indicator which has been laboratory pre-calibrated to convert differential pressure readings directly into velocities. Each point measurement should represent approximately 5 per cent of the total flow.

This method has proved reasonably satisfactory. However, it is absolutely necessary that all air be bled from the system, as air trapped in the system will cause erroneous readings. After every use the system is re-checked in the laboratory.

Current meter measurements. When flow depths become too great for personnel to operate the above equipment, standard techniques of discharge measurement with bucket type current meters are utilized. Footbridges have been constructed at all sites as attachments to highway bridges or culverts. Sounding reels mounted on dollies which traverse the upper rails of footbridges are used for this purpose.

DEVELOPMENT OF RATING CURVES AND EQUATIONS

As mentioned earlier, both upstream and downstream stages are recorded at 5-minute intervals. Each of these stages is noted at the time of making all discharge measurements.

Using data from these measurements, several methods were tried to obtain the most accurate weir rating equations. In these different methods, measured discharge was related to headwater, tailwater, or a combination of headwater and tailwater elevations. Based on our limited number of measurements, the best preliminary correlation was obtained using headwater elevations only. However, since our two years of operations were periods of relatively low flows, tailwater surface elevations seldom exceeded V-notch elevations. Submergence occurred on few occasions and insufficient measurements are available in this range to develop accurate weir ratings during submergence.

Preliminary equations developed varied from site to site and also varied at a given site for different ranges of flow depths. This is probably due to some combination of factors such as submergence, interference with flow by culvert divider walls, non-uniform channel approach, and flood plain conditions peculiar to each site.

Rating equations developed are of the standard weir type

$$Q = cH^m . \tag{6}$$

Best data correlation was obtained by plotting values of logarithm of measured discharge versus logarithm of head. Curves of best fit drawn from these plots clearly showed breaks at a specific head value. Head-discharge relations were properly described with one curve at one site, three curves at one site, and two curves at all other sites. A representative example is Station I, where

$$Q = 29.51\, H^{3.205} \text{ for } h \leqslant 1.37' \tag{7}$$

$$Q = 43.25\, H^{1.994} \text{ for } h > 1.37' . \tag{8}$$

SUMMARY

One of the most difficult problems in a watershed hydrologic study of the southeastern United States Coastal Plain is measurement of surface run-off in the low-gradient streams. Model studies of the Virginia V-notch weir resulted in recommended design specifications which provide for a compromise between control submergence and backwater ponding. Interlocking steel-sheet piling cutoff-support walls and guide walls force flow over the weir and onto a concrete water dispersal apron. Energy dissipation on the apron is provided by baffle blocks and end sills. Two digital-type stage recorders with electronic timer record upstream and downstream heads at 5-minute intervals.

Rating equations are developed through model studies for free and submerged flow. Field measurements by volumetric, current meter, and pitot-tube methods are used for verification of model results. Best preliminary ratings are obtained using headwater elevations only, since limited submergence has occurred and few discharge measurements are available.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Carter R. W. 1951. *Floods in Georgia: Frequency and Magnitude,* U.S. Geological Survey Circular 100.
2. Pride, R. W. 1958. *Floods in Florida: Magnitude and Frequency,* U.S. Geological Survey open-file report.
3. Burford, J.B.; J.H. Lillard. 1963. *High Accuracy Streamflow Measurement with Low Cost Installations.* American Society of Agricultural Engineers Paper No. 63-224, Miami Beach, Fla., 25 June.
4. Ree, W.O.; W.R. Gwinn. 1959. *The Virginia V-notch Weir.* USDA, SWCRD, ARS-41-10, Beltsville, Maryland.
5. Harrold L.L.; D.B. Krimgold 1943. *Devices for Measuring Rates and Amounts of Runoff.* USDA-SCS Technical Paper 51.

II.4 Other methods
Autres méthodes

Measurement and estimation of flood discharges

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Abstract. Flood discharge measurements may be made under extreme conditions by the use of specialized equipment or techniques. Measurement of large flows is exemplified by measurement of the Amazon, Mississippi, and Columbia Rivers. Large sounding weights are used; power equipment is used for sounding from boats, trucks, trailers, or powered cable cars. Special means may be used for stationing along a cross-section, as in the Amazon measurements.

The optical velocity meter is a device which uses the stroboscopic effect for measuring the surface velocity of a stream. This method, in conjunction with an acoustic depth sounder, was used successfully to measure flows during recent extreme floods in California.

Indirect measurements are a recourse on all sizes of streams when floods cannot be measured as they occur. Discharge is computed by post-peak measurements, through a reach of river (the slope-area method), through a bridge (the contracted-opening method), over dams, and through culverts.

MESURE ET ESTIMATION DES DEBITS DES CRUES

Résumé. La mesure des débits des crues peut être faite dans des conditions extrêmes en utilisant les équipements et techniques spécialisés. Les mesures de l'écoulement de l'Amazone, du Mississippi et de la Columbia sont des exemples de mesure des grands écoulements. On utilise des plombs de sondage de grande dimension, l'outillage mécanique pour le sondage est mis en œuvre à partir de bateaux, de camions, de remorques ou de la nacelle d'un transbordeur. On peut avoir recours à des moyens spéciaux destinés à être utilisés transversalement, comme dans le cas de l'Amazone.

Le compteur optique de vitesse est un appareil utilisant l'effet stroboscopique pour la mesure de la vitesse en surface d'un cours d'eau. Cette méthode, combinée à une sonde acoustique de profondeur, a été utilisée avec succès pour la mesure de l'écoulement des crues exceptionnelles récentes en Californie.

Lorsqu'on ne peut procéder à la mesure des crues au moment où elles se produisent, on a recours aux mesures indirectes sur les cours d'eau de toutes dimensions. Le débit est calculé à partir de mesures effectuées après passage de la pointe, dans un bief de la rivière (méthode dite de pente-surface de la section), sous un pont (méthode dite de l'ouverture rétrécie), sur les barrages et à travers des conduits souterrains. Même en l'absence de crues, les rapports niveau/débit peuvent être calculés par la méthode de remous.

MEDICIÓN Y ESTIMACION DE CAUDALES DE CRECIDAS

Resumen. Las mediciones de caudal de crecidas se pueden realizar bajo condiciones extremas mediante el uso de equipo y técnicas adecuadas. La medición de grandes crecidas tiene como ejemplo las realizadas en los ríos Amazonas, Mississippí y Columbia. Se utilizan grandes pesos de sondeo y equipo mecánico eléctrico para el sondeo desde embarcaciones, camiones, remolques o coches.

Se pueden usar medios especiales para establecer estaciones a lo largo de una sección transversal, como ocurre en las mediciones del Amazonas.

El medidor de velocidad óptico es un dispositivo que utiliza el efecto estroboscópico para medir la velocidad superficial de un curso de agua. Este método, junto con una sonda de profundidad acústica, se utilizó con éxito para medir caudales durante recientes crecidas extremas en California.

Las mediciones indirectas son un recurso en todo tipo de cursos de agua cuando las crecidas no se pueden medir conforme se producen. El caudal se calcula por mediciones *a posteriori*, en un tramo de río (método de pendiente-área) en un puente (método de la abertura contraída), en presas, y en desagües. Aún sin crecidas, las relaciones nivel-caudal se pueden calcular en cualquier emplazamiento sobre un río por el método de las curvas de remanso.

ИЗМЕРЕНИЕ И ОЦЕНКА ПАВОДОЧНЫХ РАСХОДОВ

Резюме. Измерения паводочных расходов при экстермных условиях могут быть выполнены путем использования специального оборудования и методики. Приводятся примеры измерений во время паводков на крупных реках: Амазонка, Миссисипи и Колумбия. Ииспользуются большие грузы и силовое оборудование для производства промеров с судна и грузовые автомобили, прицепы или оборудованные мотором канатные переправы (паромы). Могут быть использованы специальные средства для установки судна на вертикали, как это было при измерениях на Амазонке.

Оптический измеритель скорости — прибор, основанный на использовании стробоскопического эффекта для измерения поверхностной скорости в потоке. Этот метод в сочетании с промером глубин эхолотом успешно использовался для измерения паводочных расходов в Калифорнии.

Косвенные измерения можно применить в тех случаях, когда расходы паводков нельзя измерить во время их прохождения. Расход вычисляется по следам паводка для участка реки, используя метод — « уклон-площадь »; особенно хорошие результаты получают на мостах, через плотины, дорожные водопропускные трубы. В ряде случаев и при отсутствии паводков этим способом может быть получена надежная кривая расходов.

INTRODUCTION

The measurement of extremely high discharges on rivers poses difficult problems for the hydrographer. Extreme velocities and depths cannot be measured with the equipment ordinarily used or by routine methods. Large amounts of drift may make the task of measuring by ordinary means either impossible or extremely risky. At times, access to measuring sites is not possible during floods. On small watersheds, even though the hydrographer may be present near the time of the peak flow, the extremely rapid rise and fall of the hydrograph may prevent a reliable measurement of the discharge. Yet all these problems must be overcome if the ever-increasing needs for information on the water resources are to be met. This paper describes the equipment and methods that are being employed to obtain measurements or estimates of river flow under extreme conditions.

LARGE DEPTHS AND VELOCITIES

Under conditions of low and moderate streamflow, the discharge of many streams is measured by current meter by a man wading the stream. Where water depths or the combination of depth and velocity make it impossible to measure this way, other techniques are used. The current meter may be suspended in the water from a boat, bridge, or cableway. A sounding weight is attached below the current meter to keep it stationary in the water and to keep the suspending cable as nearly vertical as is possible. The weight also prevents damaging of the meter when the assembly is lowered to the streambed.

When the suspending cable is vertical, readings of water depth may be made directly by means of a counter on the reel on which the cable is wound. If the line is

not vertical, the depth needs to be corrected for the departure from verticality. This departure should not be too extreme, otherwise errors are introduced both because of uncertainties in sounding depths and because the position of the meter does not remain in a plane cross-section of the stream. To minimize such errors, large sounding weights are used. In United States practice, sounding weights ranging from 15 to 150 pounds (approximately 7 to 68 kg) are used to measure discharges during flood periods at most gauging stations. For exceptional depths and/or velocities, 200- and 300-pound weights (91 and 136 kg) are used. The lighter weights (up to 150 pounds) may be manipulated either by manually-cranked reels or motor-driven reels. The heavier weights require the use of powered equipment. Weights even larger than 300 pounds have been made for special purposes. For example, a 424-pound (192 kg) brass weight was developed for use on the Columbia river at Grand Coulee Dam. However, the use of such large weights tends to overstress other components of the necessary equipment. There is a practical limit to the size of the sounding weight because a heavier weight requires a large-diameter cable and this in turn increases the resistance to flow, so that the advantage of the heavier weight is lost.

Special equipment is used for making current-meter measurements of large rivers and for high velocities and depths [ref. 1]. For cableway measurements, power-operated cable cars are available for extremely long spans. One type is the self-propelled car using batteries for both car movement and reel operation. Nine of these are in use in the Columbia river basin. Another is the self-propelled car using a gasoline engine to operate a hydraulic system, which in turn operates the car and the reel.

Measuring from bridges under extreme flow conditions requires the use of specialized power equipment. Portable cranes with battery-powered reels are commonly employed. A truck-mounted or trailer-mounted crane is used for measurements on the Mississippi and other wide, large rivers. The power for the sounding reel is supplied by auxiliary batteries which may be charged by the vehicle.

When measuring from boats, usually where bridges or cableways are not available, special equipment is necessary. A tagged cable is stretched across the river, serving both to hold the boat in position at the measuring points, and to measure the horizontal distances along the cross-section. Specially designed reels are necessary for laying the cable, and special equipment is used for attaching the boat to the cable, and for suspending the meter.

Measurements on large streams such as the Mississippi river and Amazon river have been made from boats of various sizes. On the Amazon river [2, 3], a 150-foot (46 m) corvette of the Brazilian Navy was used by the United States Geological Survey in making measurements during 1963. Tellurometer equipment on ship and shore was used to determine stationing along the cross-section.

The maximum velocities that have been measured in the United States by current meter during the course of a discharge measurement range up to 24.6 feet per second (7.5 m/sec). Table 1 shows some of these record velocities. The highest velocities (22 to 24.6 feet per second or 6.7 to 7.5 m/sec) have been measured in relatively shallow streams. Most of these high point velocities were measured either at the surface or a few feet below the surface. During the flood of 17 February 1937, on the Mississippi river, a maximum point velocity of 13.3 feet per second (4.1 m/sec) was measured; the maximum depth of flow was 113 feet (34 m).

Under some conditions, drift or swift water makes it impossible to reach the bottom by means of sounding weights. Sonic sounding equipment may be used from bridges or from large boats. Under such conditions, a surface or subsurface velocity is usually measured by current meter or by visual observation.

Table 1. *Extreme point velocities measured by current meter by the United States Geological Survey.*

River	Location	Date	Sounding weight (kg)	Maximum point velocity (m/sec)	Maximum depth (metres)	Total discharge (m^3/sec)
Amazon	Obidos, Brazil	16/7/63	136	3.2	62	216 000
Mississippi	Vicksburg, Miss.	17/2/37	136	4.1	34	58 000
Columbia	Wells Dam, Wash.	8/6/67	136	5.1		12 000
North Santiam	Detroit, Oreg.	18/1/53		6.2	2.5	
North Llano	Junction, Tex.	10/2/49	45	6.4		1 060
Canadian	Newcastle, Okla.	4/5/41	45	6.7		
New River	Caperton, W. Va.	8/40	34	6.7	11	5 700
Little Colorado	Grand Falls, Ariz.	5/4/29		6.7		
Brazos	Waco, Tex.	27/9/36		6.8	12	5 800
Potomac	Chain Bridge, D.C.	14/5/32	45	6.8	9.3	4 700
Indian Creek	Council Bluffs, Iowa	15/7/50	34	7.0	2.1	
Iron	White Pine, Mich.	26/4/54		7.3		
Caney Fork	Rock Island, Tenn.	4/2/39	45	7.5	11	1 700

OPTICAL FLOWMETER

Under extreme flood conditions, surface velocities are sometimes measured by timing floating drift, and then applying coefficients to relate the surface velocity to the mean velocity in a vertical. Surface velocity is difficult to measure in this way because of difficulties in timing and because the drift does not maintain a path at right angles to the cross-section. Newly developed equipment [4] is available for accurate measurement of surface velocities. The optical flowmeter is a stroboscopic device to measure surface velocities in open channels without immersing equipment in the stream. The meter is usually attached to the railing at a bridge crossing the stream. The stroboscope has a telescope which is sighted on the water surface and a set of rotating mirrors whose speed of rotation can be varied until the apparent motion of the water surface is stopped. It is used most easily for measurements in fixed-bed channels, but may also be used in combination with a sonic sounder. During the record-breaking floods of January and February 1969 in southern California, the optical flowmeter was used to make 50 discharge measurements in canals. The velocities that were measured ranged from 5 to 50 feet per second (1.5 to 15 m/sec). The higher velocities could not have been measured accurately by any other known methods. The optical meter is useful also for the measurement of mud flows or where moving ice or other types of material carried in the water make it impossible to immerse a current meter.

INDIRECT DISCHARGE COMPUTATION

Quite frequently, because of inaccessibility, lack of manpower, or unfavourable measuring conditions, peak floods cannot be measured as they occur. Where this is so, recourse may be had to indirect measurement of the peak flow [5, 6]. The discharge is computed by post-peak measurement of several factors. Channel cross-sections or structures are measured, water-surface elevations at the time of the peak are established by a survey of high-water marks, and examination of channel conditions is made to estimate the proper hydraulic and roughness coefficients.

Discharge may be computed in this way through a reach of river (the slope-area method) [7], through a bridge or series of bridges (the contracted-opening method) [8], over dams [9], and through culverts [10]. The accuracy of such computations is dependent on the stability of the channel, the presence of good high-water marks, and the accuracy of estimating the hydraulic coefficients, and the roughness coefficient (such as the Manning n). Although such measurements are not usually of the accuracy of current-meter measurements made under good conditions, they may equal or surpass the accuracy of current-meter measurements under the unfavourable measuring conditions that commonly exist during floods. And, in most cases, they provide a value of the discharge that would not be possible to obtain by any other means.

Estimates of discharge, related to stages, may be made even in the absence of floods, or when there is little or no flow in the channels. Where well-defined hydraulic controls exist, such as a dam or a culvert, for which the proper coefficients may be estimated, a stage-discharge relation may be computed [9, 10]. All that is necessary is the measurement of the geometry of the structures, a survey of the upstream and downstream channels, and estimation of the hydraulic and roughness coefficients. Similarly, a stage-discharge relation may be computed for any site on a river by the step-backwater method [11, 12]. This requires surveys of the channel cross-sections and the stream-bed profile, and estimation of Manning's n or equivalent roughness coefficients for reaches of the channel.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Buchanan, T.J.; Somers, W.P. 1969. Discharge measurements at gaging stations. *Techniques of water resources investigations of the U.S. Geological Survey,* Book 3, Chap. A8 Washington, D.C., U.S. Government Printing Office, 65 p.
2. Oltman, R.E.; Sternberg, H. O'R.; Ames, F.C., Davis, L.C., Jr. 1964. *Amazon River investigations, reconnaissance measurements of July 1963.* (U.S. Geological Survey Circular 486).
3. Oltman, R.E. 1968. *Reconnaissance investigations of the discharge and water quality of the Amazon River.* (U.S. Geological Survey Circular 552).
4. Smith, W.; Bailey, G.E. 1962. Optical current meter. *American Soc. of Civil Engineers, Hydraulics Div. Jour.,* Separate no. 3258, September, p. 13-22.
5. Benson, M.A.; Dalrymple, Tate. 1967. General field and office procedures for indirect discharge measurements. *Techniques of water resources investigations of the U.S. Geological Survey,* Book 3, Chap. Al. Washington, D.C., U.S. Government Printing Office, 30 p.
6. Benson, M.A. 1968. *Measurement of peak discharge by indirect methods.* Geneva, World Meteorological Organization, 161 p. (Tech. Note 90, WMO-No. 225.TP.119).
7. Dalrymple, Tate; Benson, M.A. 1967. Measurement of peak discharge by the slope-area method. *Techniques of water resources investigations of the U.S. Geological Survey,* Book 3, Chap. A2.Washington, D.C., U.S. Government Printing Office, 12 p.
8. Matthai, H.F. 1967. Measurement of peak discharge at width of tractions by indirect methods. *Techniques of water resources investigations of the U.S. Geological Survey,* Book 3, Chap. A4. Washington, D.C., U.S. Government Printing Office, 44 p.
9. Hulsing, Harry. 1967. Measurement of peak discharge at dams by indirect methods. *Techniques of water resources investigations of the U.S. Geological Survey,* Book 3, Chap. A5. Washington, D.C., U.S. Government Printing Office, 29 p.
10. Bodhaine, G.L. 1968. Measurement of peak discharge at culverts by indirect methods. *Techniques of water resources investigations of the U.S. Geological Survey,* Book 3, Chap. A3. Washington, D.C., U.S. Government Printing Office, 60 p.
11. Chow, V.T. 1959. *Open channel hydraulics,* p. 265-68, New York, McGraw-Hill.
12. Woodward, S.M.; Posey, C.J. 1941. *Steady flow in open channels,* p. 94-109, New York, John Wiley.

DISCUSSION

Garg: 1. How has the coefficient for the conversion of surface and subsurface velocity been found for such high discharges? Is it determined during low discharge and applied for high discharges also?

2. Did you find any relationship between the power necessary for keeping the boat stationary and high velocity and strong winds while measuring high flood discharge?

Benson: 1. These coefficients are arrived at by studying the velocity distributions at lower stages or under similar conditions. Usually this conversion factor ranges between 85 and 90.

2. For holding a position very closely, during a 3 m/sec water velocity and a 30 miles/hour cross-wind we needed two 165 horse-power motors.

Purich: Does the 0.2 and 0.8 depth measurement necessarily apply in the case of measuring large floods where the velocity profile is not always parabolic?

Benson: No. The 0.2 and the 0.8 depth measurement is only applicable for parabolic distribution. Nevertheless, from our experience the 0.2 and 0.8 method gives rather accurate results even in large rivers.

The air-bubble method of flow measurement and its application

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Abstract. This paper describes the air-bubble method of flow measurement for use in open channels. The fundamental equation underlying the method can be derived in terms of the characteristics of flow and the movement of bubbles. Accordingly, the rate of flow is obtained as the product of the area of the flow diagram and the terminal rising velocity of the bubbles, which can be considered constant under certain conditions.

From the results of laboratory and field measurements performed in order to investigate the influence of the various factors involved, it was concluded that the terminal rising velocity of the cluster of bubbles is constant if the specified conditions of air injection are observed, and –depending on the Reynolds number of flow–it is greater in flowing than in standing water. The flow diagram is recorded and determined by a photographic method. The mean error of measurement is estimated at ± 4 to 5 per cent.

The merits and drawbacks of the method are listed with the objective of outlining the potential fields of application. Finally the practical problems of measurement are dealt with.

METHODE DES BULLES D'AIR POUR LA MESURE DES DEBITS ET SON APPLICATION

Résumé. L'étude présente une méthode de jaugeage de débit par bulles d'air pouvant être appliquée dans les canaux à surface libre. Des caractéristiques de l'écoulement et du mouvement des bulles d'air on peut déduire l'équation fondamentale suivant laquelle le débit peut se calculer comme produit de la vitesse finale de montée des bulles (pouvant être considérée constante dans certaines conditions) et de la surface du diagramme des débits.

Sur la base de mesures exécutées en laboratoire et sur le terrain pour étudier l'effet des facteurs ayant une influence, on peut constater que la vitesse finale de la montée de la masse des bulles d'air est constante – tant que les conditions d'aération prescrites sont suivies – et que cette vitesse, qui dépend du nombre Reynolds du courant, est plus grande dans l'eau en mouvement que dans l'eau au repos. Pour fixer et déterminer le diagramme des débits on utilise un procédé photographique. L'erreur moyenne de la méthode de mesure peut être évaluée à ± 4-5 %.

L'examen des avantages et des désavantages du procédé ainsi que des questions pratiques de mesures déterminent le domaine de son application.

MÉTODO DE LA BURBUJA PARA MEDICIÓN DE CAUDAL Y SU APLICACIÓN

Resumen. Se describe el método de la burbuja de aire para las mediciones de caudal en cauces abiertos. La ecuación fundamental en la que se basa el método se puede obtener en función de las características del flujo y del movimiento de las burbujas. En consecuencia, el valor del caudal se obtiene como el producto del área del diagrama de flujo y la velocidad final de elevación de las burbujas, que se puede considerar constante bajo determinadas condiciones.

A partir de los resultados de las mediciones en laboratorio y en campo, realizadas con objeto de investigar la influencia de diversos factores, se llegó a la conclusión de que la velocidad final de ascenso del grupo de burbujas es constante si se observan las condiciones especificadas de inyección de aire, en función del número de Reynolds del flujo, que es mayor en el agua en movimiento que en la estancada. El diagrama de flujo se determina y traza mediante método fotográfico.

El error medio de la medición se estima entre ± 4 y 5 %.

La ventaja del método está relacionada con la descripción del campo potencial de aplicación, una vez se han resuelto los problemas prácticos de medición.

СПОСОБ ИЗМЕРЕНИЯ РАСХОДА ВОДЫ С ПОМОЩЬЮ ВОЗДУШНЫХ ПУЗЫРЬКОВ И ЕГО ПРИМЕНЕНИЕ

Резюме. Описывается способ измерения расхода воды в открытых руслах с помощью воздушных пузырьков. Основное уравнение метода может быть выведно в функции от характеристик потока и движения пузырьков. Величина расхода получается как произведение площади скоростной диаграммы и конечной скорости подъема пузырьков, которая может быть принята постоянной при определенных условиях.

На основании результатов лабораторных и полевых измерений, выполненных с цельюисследования влияния различных факторов, было установлено, что конечная скорость подъема облака пузырьков постоаннна, еси соблюдаются особые условия ввода воздуха, и зависит от числа Рейнольдса потока, причем она больше в движущейся воде, чем в стоячей. Скоростная диаграмма устанавливается фотографическим способом. Средняя ошибка измерений оценивается в ± 4-5 %. Перечислены преимущества и недостатки метода с целью выявления возмохных областей его применения; рассмотрены практические задачи его применения.

The applicability and accuracy of currently used methods of flow measurement in open channels is adversely affected by the fact that either a considerable loss of head is necessary or measurement cannot be performed in the entire cross-section simultaneously. The latter is the case when current meters, floats, or chemicals are used. With the air-bubble method of flow measurement, an attempt is made to overcome both these difficulties.

THE THEORY OF MEASUREMENT

The integrating float method is one of several used for measuring the velocity of flow in open watercourses. It will be recalled that in this method a float having a specific gravity lower than that of water is released from the bottom. Its mean velocity in the vertical is found as the ratio of the horizontal distance travelled before it emerges at the surface and the time elapsing during the rising of the float to the surface.

One of the first to describe the integrating float method of flow measurement was the Hungarian S. Hajós who in 1904 suggested the use of air-filled balls of copper or rubber as floats. According to Kolupaila the method has been used repeatedly since that time. Floats of different solid, fluid and gaseous materials were used by Gloushkow, Bureau, Bekker, Miyagi, Pobedonoshtshev, Wells, Gloubshev and Taylor. For the method under consideration the studies by Miyagi are of interest, because he suggested the use of air bubbles of 3 to 5 mm in diameter as floats.

The theory underlying the air-bubble method of flow measurement is related to the integrating float method of current metering with the substantial difference, however, that the velocity—rather than the time of rising—is taken into consideration.

According to the fundamental concept described by Vid and Semenow the horizontal displacement l of the bubbles rising at a vertical velocity w, which may be regarded as constant in a particular watercourse, is proportional to the local horizontal velocity of flow $v(h)$, while the specific conveyance in the vertical q is

positively related to the horizontal distance L travelled by the bubble between the starting point on the bottom and its point of emergence on the surface (Fig. 1). A more detailed explanation of this statement is given in the development of the fundamental equation of the method.

Because the ratio dl/dh of the elementary horizontal and vertical displacements of the bubble are identical with the ratio of the corresponding velocities, the terms can be rearranged as follows

$$dl = \frac{l}{w} \cdot v(h)\ dh. \tag{1}$$

The distance of emergence L of a bubble rising and emerging to the surface in a vertical is found as the solution of equation 1, i.e.,

$$L = \int_0^L dl = \frac{l}{w} \int_0^H v(h)\ dh = \frac{l}{w} \cdot q \tag{2}$$

since the term $\int_0^H v(h)\ dh$ equals the specific conveyance q in the vertical and w is assumed to be constant. The correctness of this assumption will be checked subsequently.

For determining the full discharge conveyed in the cross-section (Q) it should be remembered that q and correspondingly l vary from vertical to vertical, depending on the position of the vertical s within the cross-section. The full discharge is obtained as the sum of specific conveyances i.e., taking equation 2 into consideration

$$q = \int_0^S g(s)\ ds = \int_0^S wL(s)\ ds = w \int_0^S L(s)\ ds \tag{3}$$

the term $\int_0^S L(s)\ ds$ representing the area F' of the "flow diagram" bounded by the curve $L = f(s)$ and the measuring cross-section (Fig. 1e),

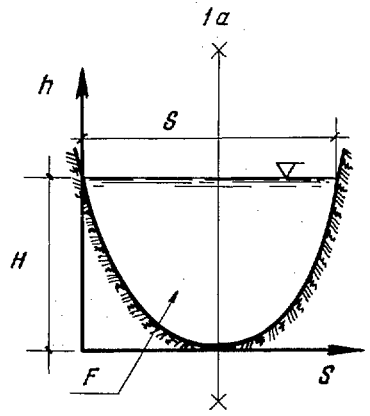
$$Q = w \cdot F'. \tag{4}$$

Discharge is consequently directly proportionate to the area of the flow diagram, and can be determined if the magnitude of w is known and F' is measured. Equation 4 may be regarded as the fundamental equation underlying the air-bubble method of flow measurement.

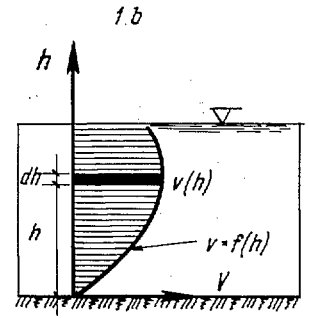
EXPERIMENTAL INVESTIGATION OF THE METHOD

The objective of the experimental investigation was to check the validity of the fundamental equation, to obtain a better understanding of the two factors and to determine a method for their measurement.

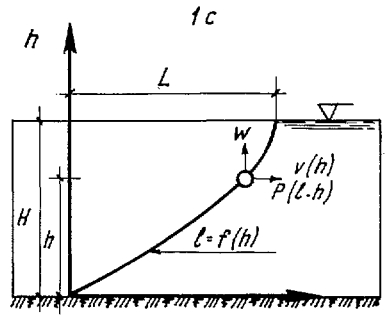
Investigations have been performed in a laboratory glass flume and in an unlined irrigation canal. The experiments have verified the correctness of the fundamental equation for both rectangular flumes and the rounded trapezoidal



Shape of the measuring cross-section



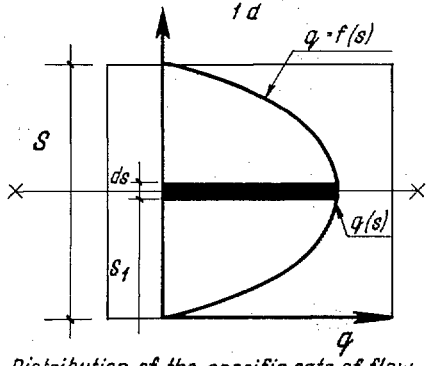
Distribution of velocity in the vertical x-x



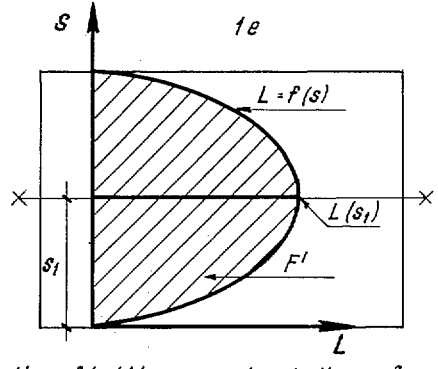
Path of bubble starting in the measuring cross-section in the vertical x-x

F = wetted area of the measuring cross-section

F' = area of the flow diagram



Distribution of the specific rate of flow in the measuring cross-section



Line of bubbles emerging to the surface L(s) and the flow diagram

Figure 1. *Characteristics of flow and bubble movement*

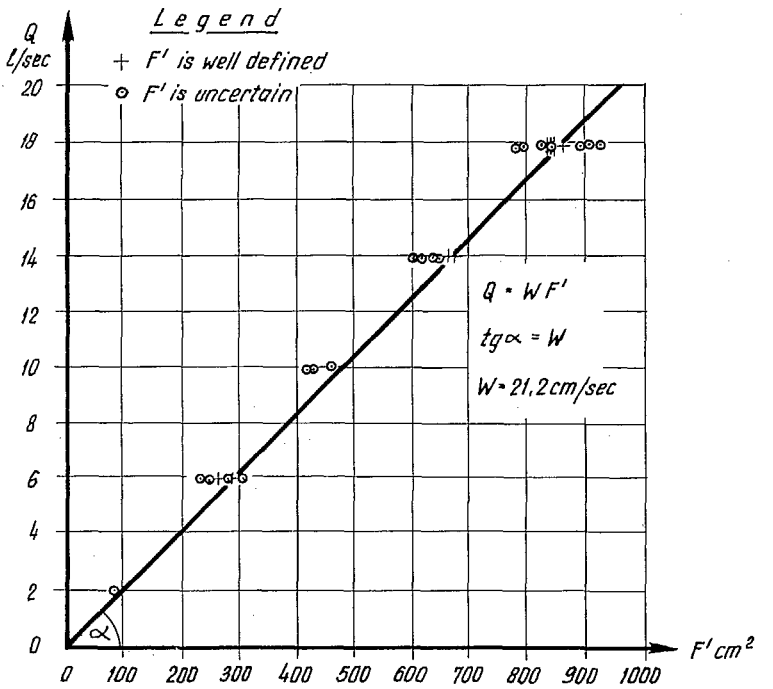


Figure 2. *Relationship between flow and flow diagram*

section of earth canals. This is demonstrated also by Figure 2 constructed from laboratory data. The correlation is, however, close enough only if the flow diagram is well defined. Flow diagrams having no clear contours cannot be used for measurement. In the first case the contours of the flow diagram are traced by the air bubbles emerging to the surface (Fig. 3), whereas in the second, large volumes of air cause undulations on the surface and the diagram becomes difficult to determine. The path of the cluster of bubbles corresponding to the case shown in Figure 3 is illustrated in Figure 4. In connexion with the photographs it should be noted that the bubbles moved during the 1/10th-second exposure.

Laboratory investigations have shown the character of the flow diagram to depend on the specific rate of air injection per nozzle on the perforated header pipe (Q_A/n, where Q_A is the rate of air flow introduced into the header pipe, and n is the number of perforations spaced in the z distance), on the mean velocity v_m of flow and on waterdepth H (Fig. 5).

The shape of the flow diagram is recorded photographically. The area can be found by the process of proportioning shown in Figure 6, where the true area F of the flow diagram is found from the known and image areas (T and T_f, respectively) of a reference rectangle of known size, and from the area F_f' of the flow diagram determined from the photograph. Attention has been directed by Vid and Semenow to the fact that an error of imaging may be introduced by the photographic method, if the asymmetry of the flow diagram is considerable. This, however, is not great enough to substantially affect the accuracy of measurement.

Another important factor in the air bubble method of flow measurement is the rising velocity w of the bubbles, or the cluster of bubbles. According to the results

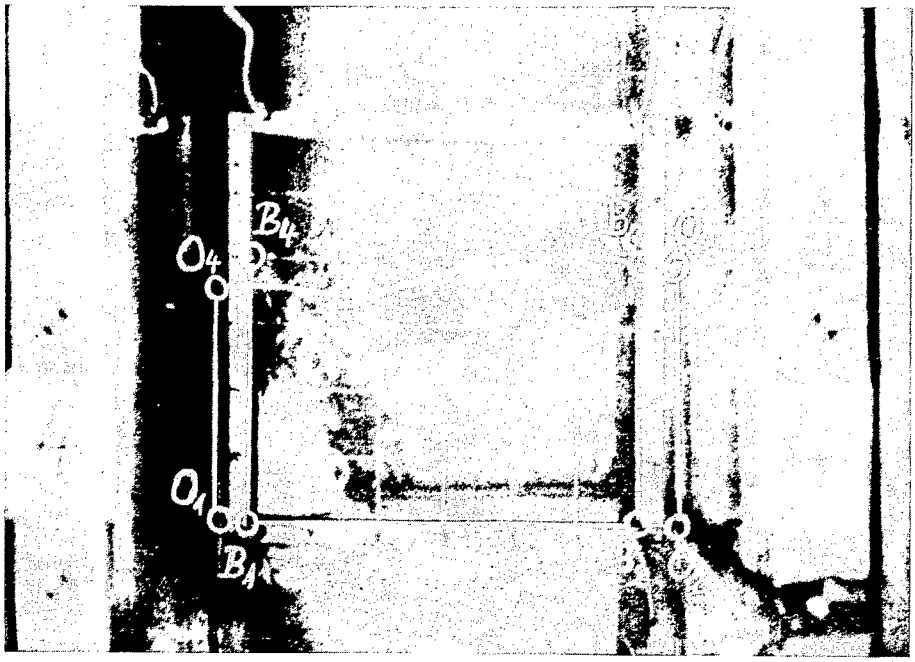


Figure 3

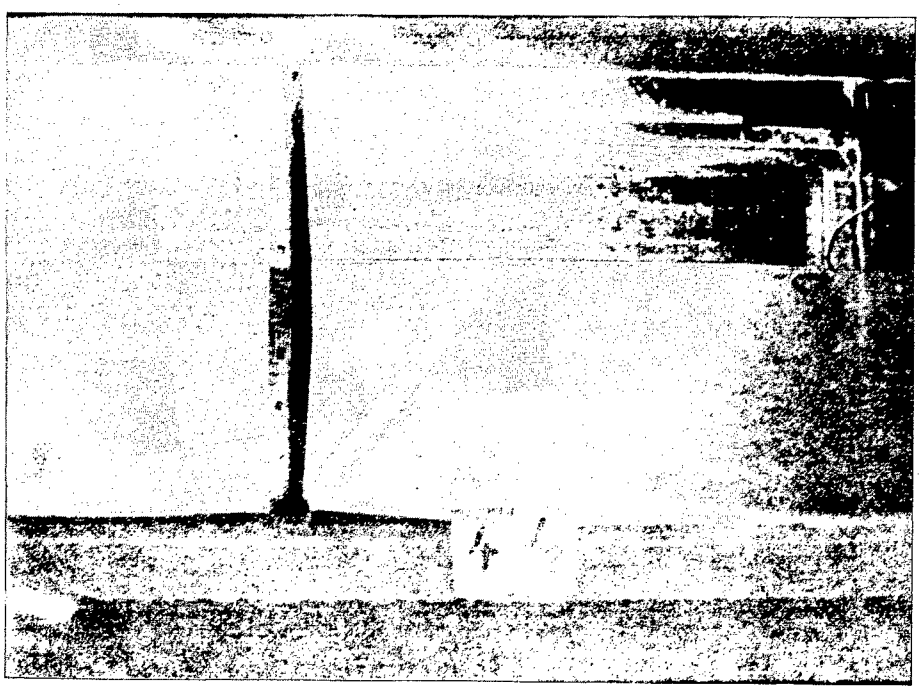
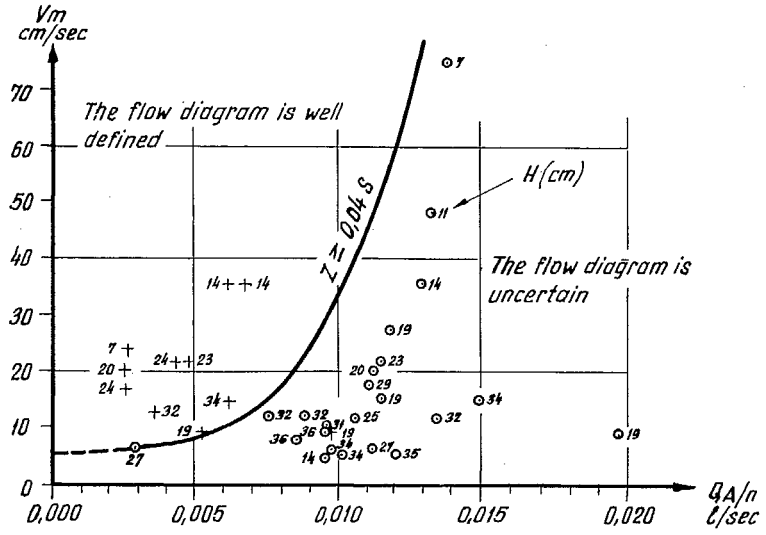


Figure 4



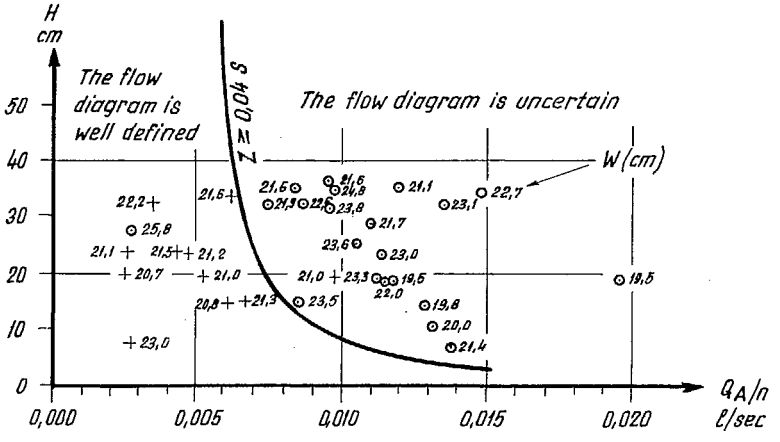


Figure 5. *Determination of the character of the flow diagram*

of investigations, bubbles starting from the bottom only accelerate over a small distance and assume thereafter a constant velocity. The average terminal rising velocity of the cluster of bubbles was found to be affected by at least fifteen independent factors, but by confining the majority of these to between given limits, w can be related to the Reynolds number of flow alone (Fig. 7).

The diagram constructed on the basis of different measurements is valid for the following factors and under the following conditions of air injection:

1. The specific rate of air flow related to a nozzle of $D = 0.5$ mm diameter does not exceed the value permissible for obtaining a well-defined flow diagram;
2. The diameter of air nozzles varies from 0.5 to 2.0 mm, depending on water depth;
3. The nozzles are spaced at $z = 0.04$ to $0.08S$;
4. The nozzles are directed upward and in the direction of flow;
5. The diameter of bubbles lies within the cluster range from $d = 2$ to 7 mm;
6. Within individual clusters $d_{max} - d_{min} \leqslant 4$ mm.

The curves shown in Figure 7 lose their validity if the above conditions are not complied with. It may still be assumed, however, that the relationship $w = f(Re)$ continues to hold for changing parameters.

Investigations have shown the terminal rising velocity of a cluster of bubbles in flowing water to be:

$$w = [1 + \beta(Re)] \cdot w_0 \tag{5}$$

where:

w_0 is the average terminal rising velocity of the cluster of bubbles in standing water. For bubbles of $d = 2$ to 7 mm diameter this velocity is 20.8 cm/sec.

$\beta(Re)$ is a factor representing the effect of turbulence, which depends—as shown in Fig. 7— on the Reynolds number of flow.

Using Figure 7 and observing the conditions listed above, the value of w required for the application of the air-bubble method of flow measurement can be determined for any open channel flow for which the Reynolds number varies from 600 to 2,100,000.

In view of the fact that the magnitude of w depends on a great number of factors, a few calibrations should be performed for the determination of the local w value where higher accuracy of flow measurement is desired.

For the field measurements performed in an irrigation canal, the perforated pipe of several coupled sections shown in Figure 8 was placed in the measuring cross-section, located 20 m downstream of the measuring bridge shown in the figure. Air was supplied from the air vessel situated on the bank. The flow diagram obtained at the rate of flow of $Q = 2.8$ m^3/sec and the reference rectangle indicated by stakes are visible in Figure 9.

If all conditions of measurement are observed, the mean error attainable with the air-bubble method of flow measurement may be estimated at ± 4 to 5 per cent as corroborated by investigations.

EVALUATION AND POTENTIAL APPLICATIONS OF THE METHOD

Interest in the air-bubble method of flow measurement may be attributed primarily to the possibility of obtaining an instantaneous figure for the rate of flow without any head loss and for the entire cross-section simultaneously, which

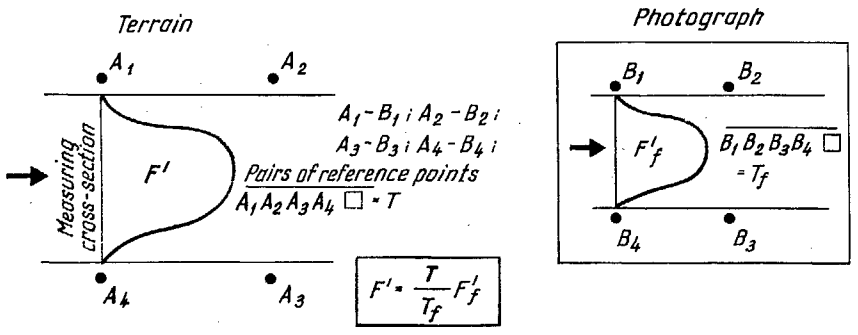


Figure 6. *Determination of the area of the flow diagram by the photographic method*

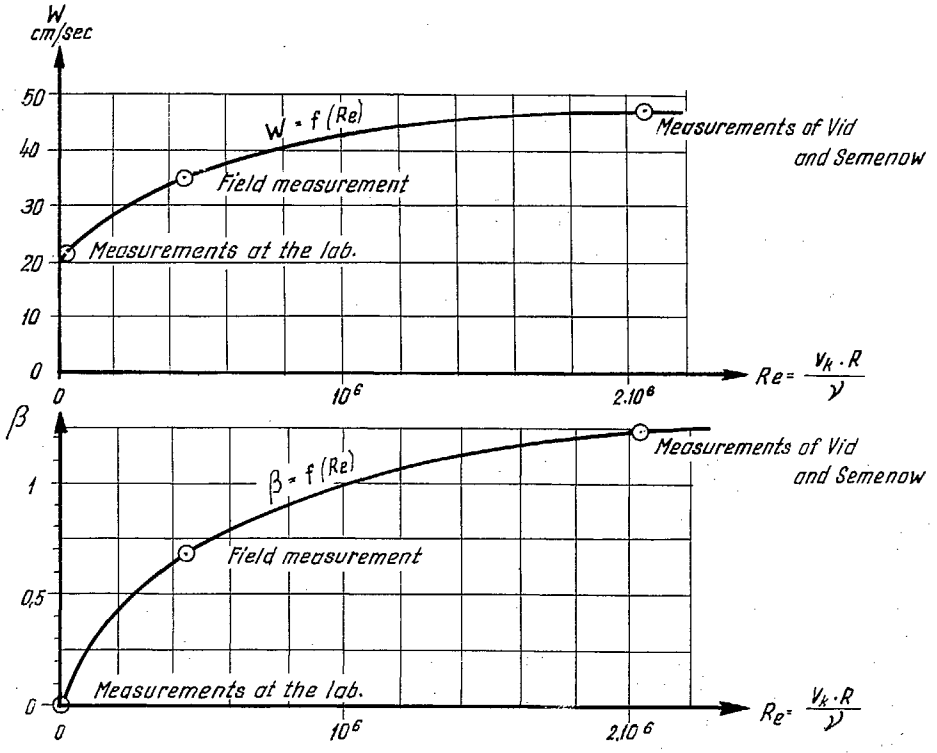


Figure 7. *W and β in terms of the Reynolds number of flow*

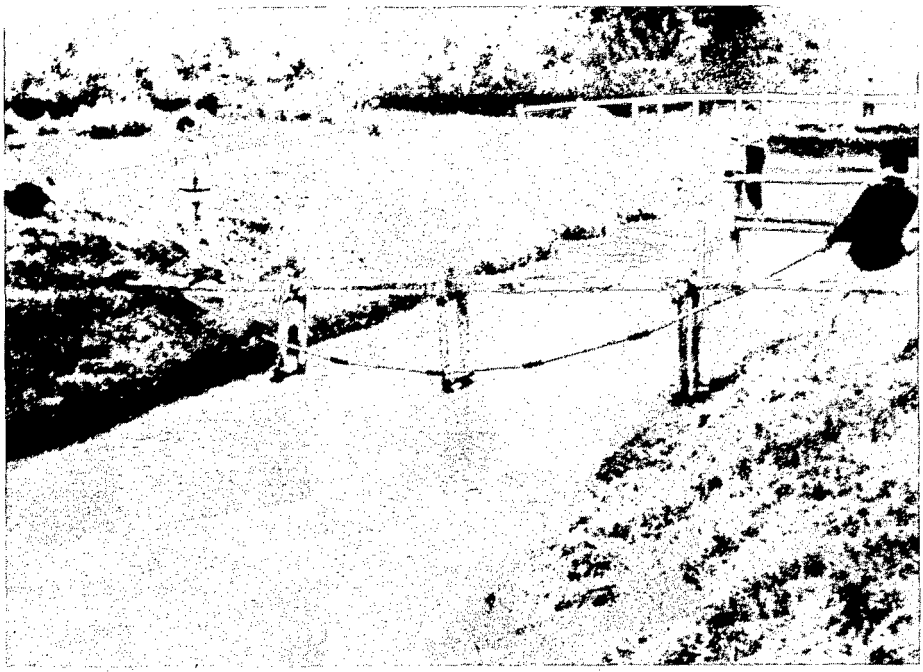


Figure 8

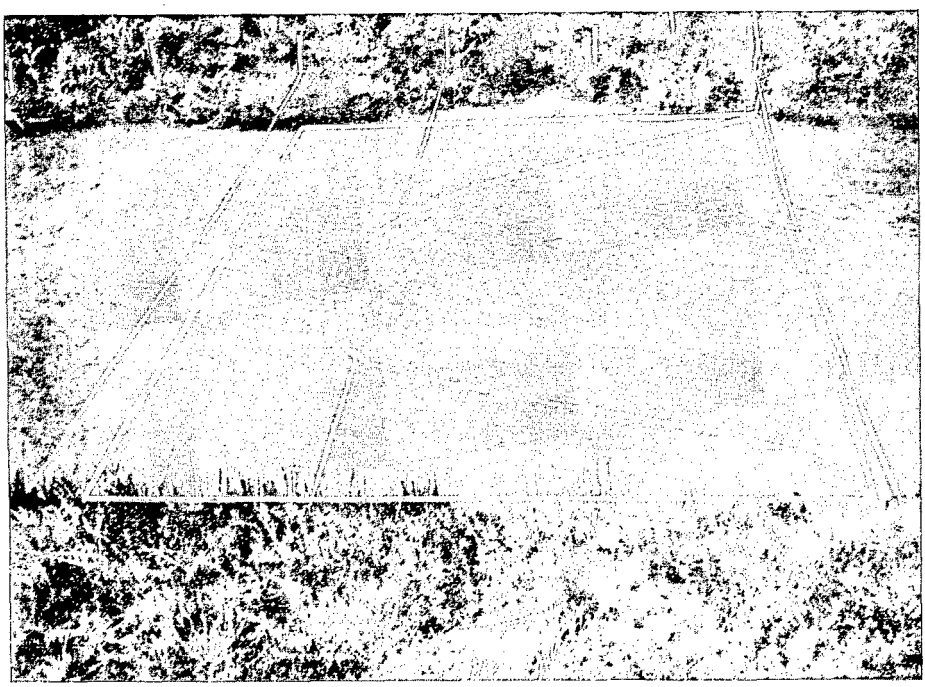


Figure 9

is advantageous especially in the case of rapidly varying, unsteady flow phenomena. No limitations are in general imposed by the method on the characteristics of the measuring cross-section. The equipment involved is inexpensive, readily portable and simple. It can be used for permanent installations, or in a portable form. Field work is rapid, simple and safe, requiring little personnel. Office processing of the field observation material is likewise rapid and simple. By this method flow measurements can be performed under conditions which would make the use of other methods cumbersome. Periods of slush-ice runs, watercourses carrying debris, inclement weather conditions and work at night using flash-lights could be mentioned as typical examples.

The method is not without a few drawbacks, the most important being that, owing to the large number of factors influencing the magnitude of w, uncertainties may be introduced. To avoid these, calibrations should be performed, limiting the applicability of portable types of equipment. Photographic materials and the training of personnel in photographic work are necessary. In wide, shallow beds, where the L_m/S ratio is low, it is impossible to use photography, owing to the resulting distortion. No simple means are as yet available for obtaining continuous flow measurements.

The application of the method, and especially its use for permanent measuring cross-sections, is recommended for irrigation and drainage canals, and also on small- and medium-size watercourses.

Besides the primary objective of flow measurement, the method lends itself to the approximate solution of a variety of other problems. The study of the distribution of flow and velocities in the profile under both laboratory and field conditions, and the hydraulic conditions in unsteady flow are but a few examples for these.

THE PERFORMANCE OF MEASUREMENTS

The use of this method in a permanent cross-section gives advantages in the accuracy attainable. The air injection pipe is installed permanently on the bottom of the cross-section, while the air supply from the bank may be from fixed or portable equipment. In a particular area, several of these cross-sections may be created and a single operator with a motor vehicle can visit them once, or even several times a day, and take the necessary photographs. The portable air supply may also be mounted on a motor vehicle. All that is required before measurement is to connect the air supply to the header pipe.

Measurements are made in the following way. The section around the proposed measuring profile should be inspected, and the profile should be located in a position where there is no change of section over the area of the flow diagram. The nozzle diameter is selected in accordance with the average depth found by profile surveying, and from the hydraulic parameters the specific rate of air flow is determined from Figure 3. In cases where $D > 0.5$ mm, the specific rate of air flow is increased in proportion to the increase of nozzle area. Nozzle spacing is computed from knowledge of surface width, which is also used to compute the nozzle number. From a knowledge of the specific rate of air flow, the total air discharge Q_A is determined to obtain a well-defined flow diagram and bubbles of the desired diameter. The capacity of the air supply should be selected accordingly. The period of air injection in the case of intermittent operation should

attain 3 to 5 minutes, but continuous operation should be given preference. Nozzles should be oriented at 45 degrees, halving the rectangle between the vertical and the direction of flow.

The hydraulic parameters of the measuring profile are used to compute the potential Reynolds numbers (extreme values) of flow, and the corresponding w values are found from Figure 7. The w value actually used in measurement is obtained by averaging these. It was found expedient to check these values initially by one or repeated calibrations.

The air injection header should be designed to permit the flexibly coupled pipe sections to follow the shape of the profile and thus to rest accurately on the bottom. The nozzles which are occasionally exposed at low stages should be carefully plugged.

Reference marks may be placed on the water-surface or in the immediate vicinity thereof. On minor watercourses a floating frame similar to that used in laboratories may be applied to advantage (Fig. 3). Photographs should be taken from above in the centreline, wherever feasible, while the centreline of the flow diagram extended to the bank should be the second alternative, as indicated in Figure 9. By evaluating the photographs, the boundary of the flow diagram should be traced along the centreline of the band of bubbles. The area F' is determined as illustrated in Figure 6. The average of two pictures should be adopted as the correct value for the area of the flow diagram.

The above principles also apply to measurements performed using a portable type of equipment. Special care should be taken in the design of the pipeline to meet the requirements of portability.

DISCUSSION

Oebius: 1. You assumed a constant velocity of air bubbles rising from a nozzle at the bottom of the water course. This assumption is not completely true. The bubbles have to be pressed out of the nozzles and therefore have an escape velocity which, during the rising, has to be transferred into rising velocity, depending only on the buoyancy of the bubbles. For accurate measurements, the nozzles would thus have to be placed at a certain distance below the bottom in order to gain a nearly constant velocity before they leave the bottom. Furthermore, the bubble stream has to establish a certain velocity distribution which is approximated to a Gaussian distribution curve.

2. According to your paper, the diameter of the air nozzles depends on the water depth. To my knowledge, it is not necessary to vary the diameter of the air nozzles according to the rising height, since the constancy of velocity in air clusters does not depend on the diameter but on the discharge through the nozzle. As a result, at increasing water depth the air discharge—and not the nozzle diameter—has to be varied. However, this causes some problems concerning the air supply. Further information on this problem is given in: Mitteilungsheft Nr. 50, *Versuchsanstalt für Wasserbau und Schiffbau Berlin,* Eigenverlag VWS, Berlin, 1970.

David: 1. In our investigations we have found that the rising velocity of bubbles is nearly constant at a very small distance, e.g. a few centimetres, from the bottom.

2. The diameter of the bubbles in our investigation was kept between 2 and 7 mm. With a given diameter of the perforation of the header pipe larger diameters of the bubbles have not been investigated by us. At very large diameters the bubbles split up into several parts. One bubble may produce 3 or 4 big bubbles and other little ones.

Purich: What are the effects of temperature on this method of measurement?

David: The rising velocity of the bubbles is more or less dependent on fifteen independent factors. One of these is the temperature. But some of these factors are negligible. We have investigated the effect of the temperature, but we have found that it is not necessary to take it into consideration.

Burz: 1. You mentioned the advantages of the method. What are the advantages with regard to the costs of the device in comparison with conventional procedures?

2. Did you take the photos from the bank or with an under-water camera?

3. What was the reason or the motive for the development and application of this method?

David: 1. We have not yet investigated the cost of this method, but it seems to me that it is not very costly. The photo-apparatus is in general use in all well-known hydraulic services. The installation of the pipe and the establishment of a good cross-section are not necessarily expensive. Using the photo is a very simple method and the accuracy of the photo in field conditions is generally quite good.

2. The best way is to take the photos from above. In field conditions this is generally not possible except from a bridge. Generally photos have to be taken from a bank, which is of course not very accurate.

3. In Hungary we have a lot of problems concerning the head losses, for which we tried to find new methods for measurement. Further, in our big drainage channels the flow may vary very quickly. If we do not have enough current meters, it takes a long time to measure the discharge. During this time the discharge varies too much. That was another reason for trying to find new methods.

Discharge measurement in open water by means of magnetic induction

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Abstract. A monitoring installation which automatically measures open water discharge and produces a discharge hydrograph is described. It is entirely without components impeding flow and the discharge cross-section is not constricted.

The underlying principle is explained, and the structure and its mode of operation are described. The installation operates on a.c., 60 Hz. Soienoids produce a homogeneous magnetic field. In the water flowing through the magnetic field, a voltage is induced which is picked up by two electrodes. The installation measures water depth and mean velocity. Multiplied by the width of the water, these values give the discharge, which is instantaneously indicated and recorded in the form of a hydrograph through impulses once every 15 min. The measured values can be stored and automatically averaged to give mean daily and mean monthly discharges.

MESURE DES DEBITS DANS UN ECOULEMENT A SURFACE LIBRE AU MOYEN DE MESURES D'INDUCTION MAGNETIQUE

Résumé. L'auteur présente une nouvelle installation qui fonctionne automatiquement et qui mesure le débit des cours d'eau et enregistre les variations du débit.

L'installation ne comprend pas de parties mobiles ou de composantes ralentissant le débit. La section mouillée est sans contraction.

L'article explique le principe de travail et décrit la construction et le mode de fonctionnement de l'installation. Des électro-aimants produisent un champ magnétique homogène. Dans le liquide s'écoulant à travers ce champ, l'induction fait naître une tension qui décèlent deux électrodes. L'installation mesure la profondeur de l'eau et la vitesse moyenne. Ces valeurs multipliées par la largeur du chenal donnent le débit ; il est indiqué immédiatement et enregistré comme hydrogramme, au moyen d'impulsions, tous les quarts d'heure.

Ces valeurs peuvent être mises en mémoire et traitées automatiquement pour fournir les débits moyens journaliers et mensuels.

MEDICIÓN DE CAUDAL EN LÁMINA LIBRE POR MEDIO DE INDUCCIÓN MAGNÉTICA

Resumen. Se describe una instalación piloto que automáticamente mide el caudal en lámina libre y dibuja el hidrograma. Carece en absoluto de partes móviles o componentes que impidan el proceso de descarga y la sección transversal de caudal no tiene constricción.

Se explica el principio básico y se describe la estructura y su modo de funcionamiento. La instalación funciona con corriente alterna de 60 ciclos. Los solenoides producen un campo magnético homogéneo. En el agua que fluye a través del campo magnético, se induce un voltaje que se recoge en dos electrodos, la instalación mide la profundidad del agua y la velocidad media. Multiplicados por la anchura de la lámina, dichos valores dan el caudal, que se indica instantáneamente, y se registra en forma de hidrograma por medio de impulsos que actúan una vez cada quince minutos. Los valores de medición se pueden almacenar y obtener la media automáticamente, para dar los caudales medios diarios y mensuales.

En la exposición de instrumentos se presenta un modelo de trabajo.

ИЗМЕРЕНИЕ РАСХОДА В ОТКРЫТЫХ ПОТОКАХ МЕТОДОМ МАГНИТНОЙ ИНДУКЦИИ

Резюме. Описывается установка, автоматически измеряющая расход воды открытого потока и выдающая гидрограф стока. Она полностью лишена движущихся частей и компонентов, препятствующих измерению расхода. Поперечное сечение, в которм измеряется расход, не сужается. Объаснен принцип, лежащий в основе способа и описана конструкция и способ ее действия. Установка действует на переменном токе, 60 периодов. Соленоиды создают однородное магнитное поле. В воде, текущей через это магнитное поле, создается вольтаж, который улавливается двумя электродами. Установка измеряет глубину воды и среднюю скорость. Будучи умноженными на ширину потока, эти значения дают расход, который мгновенно указывается и записывается в форме гидрографа посредством импульсов, действующих каждые 15 минут. Измеряемые величины могут быть накоплены и автоматически осреднены, чтобы получить средние за день или месяц значения расходов. Действующая модель демонстрировалась на выставке инструментов.

In river-flow measurements, efforts are made to measure the discharge of open waters continuously. A suitable experimental installation, using magnetic induction, has recently been set up in Lower Saxony. Its principle may be described as follows: if an electrical conductor moves through a magnetic field so that the magnetic force lines are cut, an induction voltage is generated in the electrical conductor proportional to the latter's velocity of motion. Running water is an electrical conductor, but the way in which it moves through the magnetic field differs from that of other electrical conductors. It may be seen as a bundle of many electrical conductors of differing velocities. In each individual conductor of this bundle, an induction voltage is generated in accordance with the conductor's velocity. The sum of the individual voltages can be picked up by electrodes at the banks; it is proportional to the mean flow velocity of the water.

If measuring instruments have to be immersed in the water, they may trap floating bodies (leaves, grass, wood, ice, etc.) and the measured result will be incorrect. In the case of the new measuring installation described here, no parts project into the wetted cross-section and floating bodies can no longer influence the measurements. Thus, the prerequisite conditions for continuous automatic measurement are now fulfilled.

The magnetic measuring installation is, however, sensitive to the influence of iron if the latter's distance from the installation is less than 1 m. Since all bridges contain iron, measurements cannot be taken under bridges; yet favourable measuring arrangements can be made close to bridges.

The highest positive induction voltage in water occurs when the latter flows parallel to the axis of the river in the downstream direction. The highest negative induction voltage occurs when it flows in the opposite direction. Oblique flows produce lower induction voltages according to their flow angle. This dependence of induction on flow direction is very valuable from the technical point of view because turbulent flow can now be measured as exactly as laminar flow; even eddies can be measured. In the latter case, there are negative voltages in the backflow section which are automatically subtracted. This was proved convincingly in an intake section by means of a strong artificial eddy within the range of the magnetic field. The discharge was reduced so that the pointer indicated a much lower value and finally dropped almost to zero. Visible backflow at the banks and

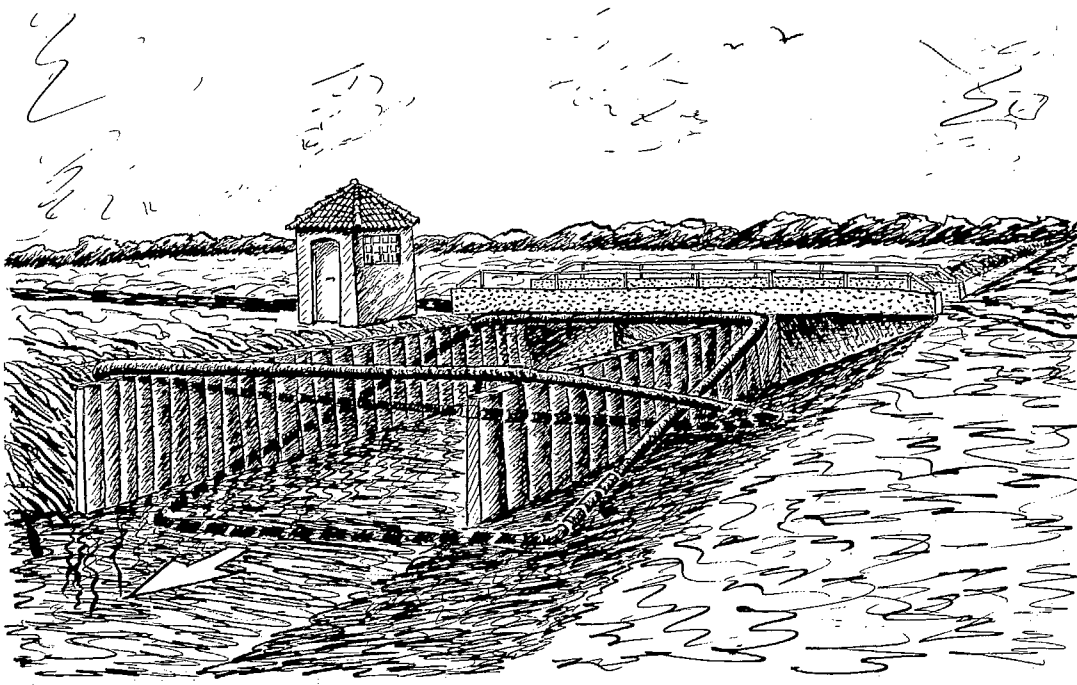


Figure 1. *Discharge measuring installation in a drainage channel of Lake Steinhuder Meer*

invisible backflow below the water surface are automatically measured by the new installation.

Any new installation of this type must be calibrated with the aid of current meters and depth soundings. The calibration will be valid as long as the cross-section at the measuring point remains unchanged.

The current consumption costs arising from continuous measurement can be reduced to about one-tenth or one-fifteenth if only brief measurements are taken at regular intervals. If this is done, the measuring sequence must be so harmonized with the selected recorder speed that the records will still show continuous stage and discharge hydrographs.

Our experimental installation has been set up in a drainage channel of Lake Steinhuder Meer, at Bad Rehburg (Fig. 1). A measurement section 6 m long and 3.75 m wide was prepared, using vertical wooden sheet piling, as a continuation of an existing concrete bed the clear width of which was the same as that of the measurement section. The bottom was stabilized by concrete filling. The measurement section is framed on all sides by two electromagnets in the form of a tilted cross.

The measuring process consists of two separate measurements. For *depth measuring* (Fig. 2), an automatic controller activates the electromagnets so that a horizontal magnetic field M_T is produced in the water. The bank electrodes pick up a voltage U_T which depends on the water depth T or on the water level at the gauge. The U_T voltage is amplified and directed into the compensator. The current

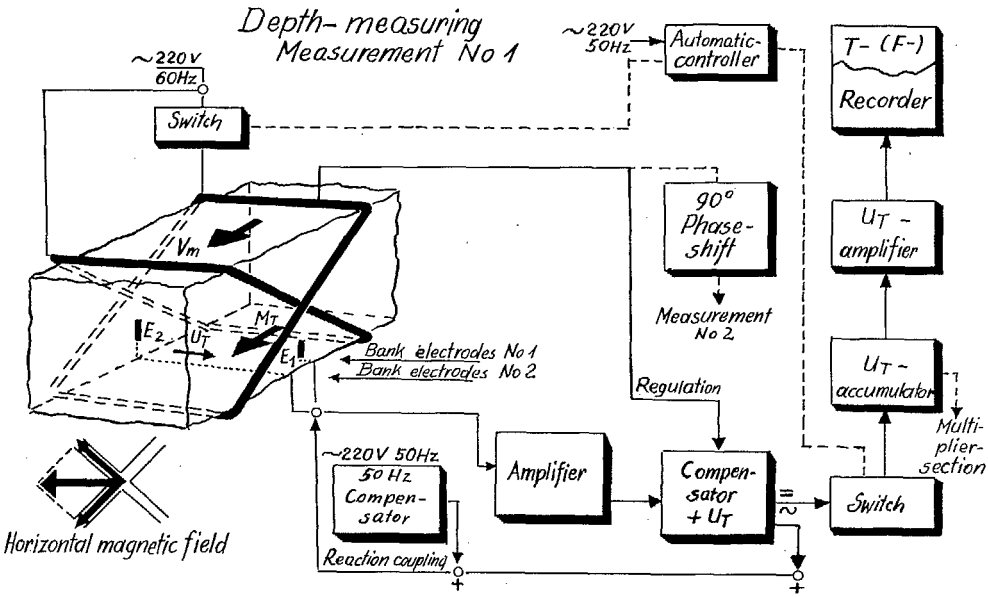


Figure 2. *Block diagram of circuit for depth measurement*

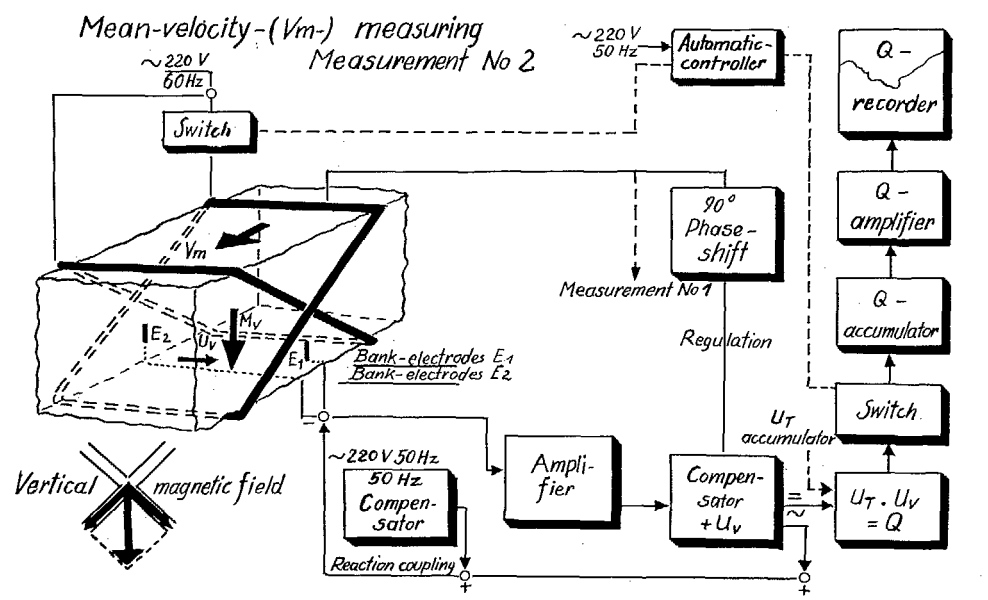


Figure 3. *Block diagram of circuit for velocity measurement*

is converted from a.c. into d.c. and stored in the accumulator. At this point, stage information can be obtained by telephone, and the stage hydrograph can be recorded.

This is followed immediately by the *velocity measurement* (Fig. 3). The electromagnets build up a vertical magnetic field M_v in the water, where voltage U_v is induced which is proportional to the mean velocity v_m. The amplified U_v voltage is fed into another compensator. The alternating current is then converted into direct current and directed to the multiplier section. Before the U_T voltage is picked up from the U_T accumulator, the flow velocity can be recorded.

By multiplication of these two measured values and of factor B (= width), we obtain the discharge

$$Q = U_v \cdot U_T \cdot B,$$

corresponding to

$$Q = v_m \cdot F.$$

Now, the discharge can be printed or transmitted, or the discharge hydrograph can be recorded. It is possible to store the Q of the individual measurements of the day, and even of the month, and to determine their mean value, thus automatically providing the required information on mean daily or monthly discharge. In previous systems final discharge values were obtained in a cumbersome way, especially in cases of weed growth where a fluctuating discharge retardation has to be taken into account.

Part of the discharge and stage hydrograph at a time of artificial interference with the discharge process is shown in Figure 4 to demonstrate the possibilities of the induction method.

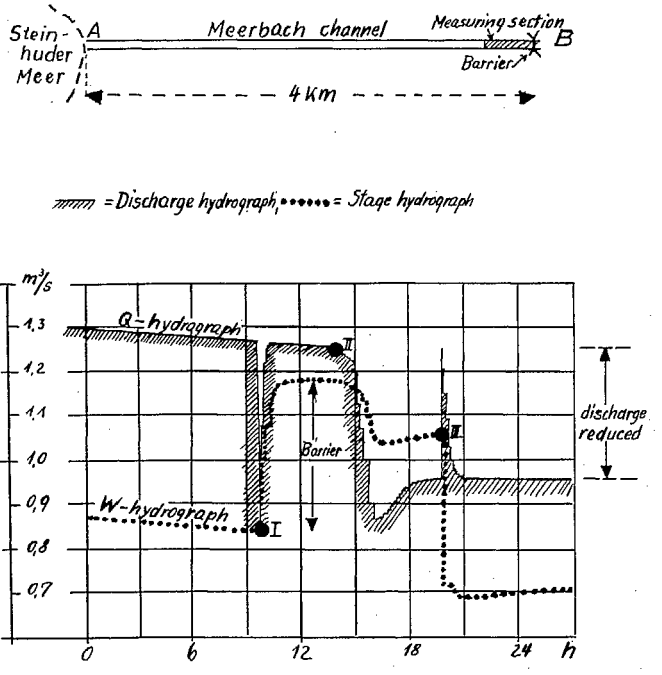
In experiment I a 40-cm-high barrier was erected at the end of the 6-m measurement section. The stage hydrograph rose. After evening out, the discharge hydrograph indicated the value that had prevailed before the erection of the barrier. Thus this is a test for the applicability of the method under conditions of retarded discharge; unimpeded, naturally retarded and artificially retarded discharge are measured with equal accuracy.

In experiment II the outfall of the outlet structure, located 4 km upstream from the measurement section, was raised by 10 cm so that the discharge was reduced to about half the normal value. Both hydrographs reacted accordingly.

In experiment III, the barrier erected for experiment I was removed again. The water level dropped, while the discharge reacted only briefly to the removal of the barrier.

The experimental installation was operated, at first, by means of a.c. 6 V, 50 Hz obtained through a transformer from the 220 V grid. However, the voltage suffered from interference from stray voltages of the same order and of frequencies of 50 Hz (or a full multiple thereof). Hence the frequency was raised to 60 Hz to facilitate screening of the interference.

It is interesting to note that a state of flow is possible in the channel which escapes measurement by the new installation. When the water flows at a velocity beyond the critical value, causing a sudden drop of the water level which advances in the direction of the flow, the indicated velocity will, during the moment at which the water level jump travels through the magnetic field, drop almost to zero. At this state of flow, the energy of the supercritical flow is partially



At the time:

I. A 40 cm high barrier was erected at the end of the measuring section.

II. The overfall of the outlet structure located 4 km upstream from the measuring section was raised by 10 cm with the result that the discharge was reduced to about 50 per cent the normal value.

III. The barrier erected at time I is removed again.

Figure 4. *Discharge and stage hydrograph during artifical interference in the discharge process*

consumed by the building up of a higher water level at the other side of the water level jump. Furthermore, when the flowing water drops on to the water cushion of larger cross-section, the direction of the flow is diverted through up to 90°, so that the direction of the velocity is no longer perpendicular to, but parallel to, the magnetic force lines, so that only a few of them are cut. At this moment the requisite conditions for velocity measurement are no longer fulfilled.

Fortunately, this artificially produced state of flow occurs very seldom in natural waters. Nevertheless, this change in flow state must be examined thoroughly. The abnormal conditions it involves may reveal unknown facts that may be of value in connexion with the measuring system.

As far as is known at present, installations of the size of Rehburg will not be much more expensive than a good recording gauge installation of conventional design. Larger installations will, of course, cost more, but the costs do not increase in direct proportion to the cross-section. For watercourses overflowing their banks regularly, a combination-type installation can be set up, consisting of a continuously measuring mean water installation and a complementary installation for the overflowing water which is actuated during the flood period, but turned off automatically after the flood wave has passed.

The induction measuring method will lead to new ways of surface discharge determination and will considerably reduce the work involved. Once the network of gauges consists to a large extent of such measuring installations, only a fraction of the work presently required will be needed. The time-consuming determination of the weed-retarded discharge will then be superfluous. The new measuring method will probably open new ways to measurement in low and high tidal currents and in tidal sluiceways.

BIBLIOGRAPHY / BIBLIOGRAPHIE

by R. W. Herschy

Anon. 1964. *Mesure de vitesse dans les eaux naturelles. Méthode électromagnétique.* Centre de Recherches et d'Essais de Chatou.

Balls, B.W.; Brown, K.J. 1959. The magnetic flowmeter. *Transactions of the Society of Instrument Technology,* June.

Bowden, K.R. 1956. The flow of water through the Straits of Dover. *Phil. Trans. Royal Society,* no. 953, vol. 248, p. 517-51.

Cox, R.A. 1956. *Measuring the tidal flow in the River Humber.* The Dock and Harbour Authority.

Harrison, D.N. 1969. An experimental electromagnetic meter. *Institution of Water Engineers Symposium on River Flow Measurement.*

Hutcheon, I.C. 1964. Electrical characteristics of the magnetic flow detection. *Head Instrument Engineer,* vol. 4, no. 1.

Longuet Higgins, M.G. 1949. The electrical and magnetic effects of tidal streams. *Monthly notices of the Royal Astronomical Soc.,* vol. 5, no. 8.

Paraday, M. 1832. *Phil. Trans. Royal Society,* p. 175.

Remenieras, G.; Hermant, C. 1954. Mesure electro magnétique des vitesses dans les liquides. *La Houille Blanche,* Special B.

Shercliff, J.A. 1962. *The Theory of Electromagnetic Flow Measurement.* Cambridge University Press.

Young; Gerrard; Jeyons. 1920. *Phil. Magazine,* vol. 40, p. 149.

DISCUSSION

Herschy: The advantage of this method is the output of continuous discharge which can be used for measurement in rivers susceptible to weed growth. Comments on the minimum velocity so far measured, the value of the input current and voltage applied to the system and on the interference from the earth's magnetic field or from magnetic storms during the operation of the stations would be appreciated.

Uhlendorf: Have attempts been made to apply the method to other than rectangular profiles?

Gils: Dr. Kirsch has investigated other arrangements of magnetic fields. He has found that the arrangement demonstrated in the model is obviously the best one. Tests in the laboratory deviated only by some 2 per cent. At present the method has been applied only in rectangular profiles such as always occur near bridges, but at a later stage other profiles will also certainly be investigated.

Kinori: The electric conductivity of the water is a function of the salt content and sediment content. How could these variations be taken in consideration?

Gils: The conductivity of the water is important. If it did not exist, the method would be useless. The conductivity itself, however, does not influence the results, but only changes in the conductivity. The whole field is still under investigation, but the first results are encouraging.

Gruber: Is it possible that an encrustration of the electrodes could influence the results?

Gils: We have tried to choose a material which is very resistent to encrustation.

Smoot: We have experimented in the United States with this method and it seems that the electrical conductivity of the water has no effect on the output of the electromagnetic flowmeter. Only the potential is determined by the strength of the magnetic field.

Graze: Over the past few years we, at the University of Melbourne, Australia, have been interested in the accuracy of electromagnetic flowmeters installed in round conduits, especially with respect to uniform or non-uniform magnetic fields and with the effects of disturbance on the output of these meters. Our investigations have indicated that a great future can be foreseen for this electromagnetic induction method. My main comment, however, concerns the statement that the conductivity of the fluid is immaterial. From my own experience this is not correct, especially when considering the extreme case of a conductivity "zero" for which the meter would nevertheless give a reading (oil is thus generally not measured by this method).

The effects of the variations of conductivity can, however, be minimized by appropriate electrical design with respect to the multiplication factors of the potential ratios having regard to the reference potential. Changes of the output of the meters as a result of the different conductivities can then be expected to remain well within 1 per cent, provided the conductivity exceeds 50 μmohs. In general, 20 μmohs is regarded as the lower threshold of conductivity. It should be noted that additives, such as anti-freeze, are very useful admixtures for raising the conductivity.

Volker: Six years ago in the Netherlands we took a measurement in a tidal river (Wester-Schelde) using the natural magnetic field of the earth. The induced voltage actually was similar to the tidal movement. The conversion of electric values into discharge was the main difficulty due to the great conductivity of the soil.

Accuracy and rationalization of river discharge measurements

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Abstract. In measuring discharge by current meter, the so-called errors of the method, both random and systematic, are of primary importance; the errors of depth and velocity measurements should be considered first of all. The latter error is due to turbulent pulsation of velocity and depends slightly on the instrumental error.

Areal averaging of depth and velocity from a series of verticals decreases the error of discharge measurements. It allows a certain increase of error in velocity measurement without a considerable increase in the error of discharge. It also permits a reduction in the duration of current meter exposure at a given point, which in turn accelerates the discharge measurements. This averaging in space and time is most useful for the integration methods of discharge measurements, particularly for the integration of velocities down the vertical. Sufficient accuracy is provided by the use of a current meter with a contact every other revolution, which eliminates the influence of marginal effects.

PRÉCISION ET RATIONALISATION DU JAUGEAGE DES DÉBITS DES RIVIÈRES

Résumé. Dans le jaugeage au moulinet, les erreurs dues à la méthode, ou les erreurs accidentelles et systématiques, ont une grande importance. Parmi ces erreurs, il faut tenir compte en premier lieu des erreurs de mesure de profondeur et de vitesse du courant d'eau. Cette dernière erreur est due à la pulsation turbulente de la vitesse et, jusqu'à un certain point, à l'erreur instrumentale.

Le calcul de la vitesse et de la profondeur moyennes pour la totalité des verticales réduit l'erreur de la mesure du débit de l'eau. Il s'ensuit que l'erreur dans la mesure de la vitesse peut croître quelque peu sans que cela entraîne un accroissement considérable de l'erreur sur le débit. Ainsi, on peut réduire la durée de l'exposition du moulinet dans les points de mesuré sur les verticales et accélérer le jaugeage. L'effet du calcul des valeurs moyennes sur la rapidité d'exécution du jaugeage se manifeste notamment dans les jaugeages par intégration, particulièrement par l'intégration de la vitesse sur une verticale. Une précision suffisante de la mesure est obtenue en utilisant un moulinet avec contacts à chaque révolution, ce qui élimine l'influence de l'effet de bord.

PRECISIÓN Y RACIONALIZACIÓN DE LAS MEDICIONES DE CAUDAL EN LOS RÍOS

Resumen. En las mediciones de caudal por un medidor de corriente, los llamados errores del método, aleatorios y sistemáticos, son de importancia primaria; los errores en las mediciones de profundidad y velocidad deben ser considerados en primer lugar y el último de estos errores se debe a la pulsación turbulenta de la velocidad, y depende ligeramente del error de los instrumentos.

La obtención de valores medios para la profundidad y la velocidad para una combinación total de verticales disminuye el error en las mediciones de caudal. La circunstancia indicada permite suponer un cierto aumento en el error de las mediciones de caudal, y permite disminuir la duración de la exposición del medidor de corriente en los puntos de la vertical, que a su vez permite la aceleración de las mediciones. El efecto de la obtención de medias en espacio y tiempo, desde el punto de vista de la aceleración de mediciones, es más ventajoso en los métodos de integración de mediciones de caudal, particularmente en la integración de velocidades a lo largo de la vertical. Se logra una precisión suficiente en las mediciones por aplicación de un medidor de corriente con un contacto a cada revolución sucesiva, que elimina la influencia de los efectos marginales.

ТОЧНОСТЬ И РАЦИОНАЛИЗАЦИЯ ИЗМЕРЕНИЯ РАСХОДОВ ВОДЫ НА РЕКАХ

Резюме. При измерени расходов воды гидрометрической вертушкой наибольшее значение имеют так называемые ошибки метода, случайние и систематические, среди которих прежде всего следует учитывать ошибки измерения глубин и скоростей. Последняяя определяется турбулентной пульсацией скорости и, в меньшей степени, инструменатальной погрешностью.

Пространственное осреднение скорости и глубины совокупности вертикалей снижает погрешность измерения расхода. Указанное обстоательство дает возможность без существенного увеличения ошибки определения расхода допустить некоторое увеличение погрешности определения скорости, что позволяет сократить время выдержки вертушки в точках вертикали и за счет этого ускорить измерение расхода. Эффект пространственно-временного осреднения, с точки зрения усорения измерительных операций, наилучшим образом используется при интеграционных методах измерения расходов воды, в частности, при интеграции скоростей на вертикали. Достаточная точность измерения обеспечивается применением вертушки со контактом через один оборот, что исключает влияние краевых эффектов.

Discharge is computed by indirect measurements comprising a number of procedures performed at different times and at different points. The time and space basis of discharge measurements allows a reduction of the effect of individual errors, i.e. random errors (systematic errors are not amenable to a statistical reduction and must be compensated for by corresponding corrections).

Random errors of discharge measurement by the "velocity-area" method comprise a combination of errors, including the error of measurements of time intervals by a stop-watch, the error caused by the pulsation of flow, a random component of the error contained in the formula for estimating the mean velocity on the vertical, the error of depth measurements due to bottom unevenness and waves on the water surface and the error arising from having an insufficient number of verticals for velocity and depth measurements.

Probability methods, usually applied in the theory of errors for the assessment of random errors, can be used provided that the errors are mutually independent. This condition is satisfied for all the above-mentioned errors except that of discharge measurements resulting from the inaccurate position of the verticals. This error increases the cross-sectional area in one partial section and decreases it in the adjacent one. The mutual dependence of these errors decreases the influence of the displacement of the verticals on the accuracy of discharge measurements. Thus the error of the determination of the distance between the verticals in the analysis of the discharge measurement accuracy may be neglected.

If discharge Q is computed by the formula

$$Q = k \cdot v_1 \cdot w_1 + 0.5\,(v_1 + v_2) \cdot w_2 + \ldots + \\ + 0.5\,(v_{n-1} + v_n) \cdot w_n + k \cdot v_n \cdot w_{n+1} \qquad (1)$$

where v = mean velocity on verticals; w = partial cross-sectional area between verticals then its random error may be presented as follows:

$$\sigma_Q^2 = k^2 \cdot \sigma_{v_1}^2 \cdot \left(\frac{\partial Q}{\partial v_1}\right)^2 + k^2 \cdot \sigma_{w_1}^2 \left(\frac{\partial Q}{\partial w_1}\right)^2 + \sigma_{(v_1+v_2)}^2 \left(\frac{\partial Q}{\partial (v_1 + v_2)}\right)^2 +$$

equation continued overleaf

$$+ \sigma^2_{w_2} \left(\frac{\partial Q}{\partial w_2}\right)^2 + \ldots + \sigma^2_{(v_{n-1}+v_n)} \left(\frac{\partial Q}{\partial (v_{n-1}+v_n)}\right)^2 +$$

$$+ \sigma^2_{w_n} \left(\frac{\partial Q}{\partial w_n}\right)^2 + k^2 \sigma^2_{v_n} \left(\frac{\partial Q}{\partial v_n}\right)^2 + k^2 \sigma^2_{w_{n+1}} \left(\frac{\partial Q}{\partial w_{n+1}}\right)^2. \quad (2)$$

Here σ with indices Q, w and v are mean square random errors of discharge, partial cross-sectional areas between verticals and mean velocity on the verticals, respectively. The shape of marginal cross-sections is taken into account by coefficient $k \leqslant 1$.

Partial derivatives of discharges may be presented in the following way :

$$\frac{\partial Q}{\partial v_i} = \frac{Q_i}{v_i}; \quad \frac{\partial Q}{\partial w_i} = \frac{Q_i}{w_i}. \quad (3)$$

We may suppose that relative errors of measurements of velocities σ_{ov} and area elements σ_{ow}, respectively, are equal in all the partial cross-sections, while mean velocities on adjacent verticals differ slightly. Then, substituting equation 3 into 2, for absolute and relative random errors of discharge measurements it is possible to obtain

$$\sigma_Q = \sqrt{(0.5\,\sigma_v^2 + (2Q_1^2 + Q_2^2 + \ldots + Q_{n-1}^2 + Q_n^2 + 2Q_{n+1}^2)} \quad (4)$$

$$\sigma_{oQ} = \sqrt{\frac{(0.5\,\sigma_{ov}^2 + \sigma_{ow}^2)\,(2Q_1^2 + Q_2^2 + \ldots + Q_{n-1}^2 + Q_n^2 + 2Q_{n+1}^2)}{Q^2}} \quad (4')$$

The structure of these equations establishes that the random error of discharge is also determined by the distribution of elementary discharges over the river width, as well as the errors of measurement of cross-sectional areas between the verticals, mean velocities on the verticals and the number of the verticals; this distribution may be characterized by coefficient

$$\eta = \frac{(n+1)\sum_1^{n+1} Q_i^2}{Q^2} \quad (5)$$

(terms 1 and $n + 1$ are supposed to be multiplied by 2).

Thus the formula of relative random error of discharge measurements may be written as follows:

$$\sigma_{oQ} = \sqrt{\frac{\eta}{n+1}(0.5\,\sigma_{ov}^2 + \sigma_{ow}^2)}. \quad (6)$$

This formula takes into account a combined display of random elementary errors and non-random factors of the flow structure and method of measurements, presented by parameter η. The values of η depend on the number of verticals and the channel shape. With an even distribution of discharge over the river width $\eta = 1$. In most of the cases coefficient $\eta > 1$; its values are shown in Table 1.

Table 1. *Values of η for various channel shapes*

Channel shape	η
Approaching a rectangular shape (canyons of mountain rivers, canalized rivers)	1.1–1.5
Stable parabolic	1.5–1.7
With flood plains and without flood plains with uneven distribution of depths	1.7–3.0

It follows from formula 6 that errors of velocity measurements are less important than the errors of the estimation of partial cross-sectional areas between verticals. The duration and complexity of discharge measurements, however, finally depend on the accepted methods and technique of velocity measurements. Consequently, to accelerate the estimation of discharge, which will increase the over-all accuracy of stream flow inventories, it is essential to reduce the time spent in determination at each site of the velocity field. This involves the transference of instruments over the width and depth of the stream, plus the duration of the current meter exposure at the points of velocity measurements. Without regard to the first type of time expenditure, which depends on the type of equipment used, we shall consider the possibility of reducing the time of the current meter exposure.

The error of measurement of the partial cross-sectional areas between the verticals may be assumed to be known. In the case of a stable channel in lowland rivers $\sigma = 2$-4 per cent. The problem of further analysis is reduced to determining the effect of time of exposure on the accuracy of velocity measurements. This problem may be solved by the theory of random functions.

With steady, uniform flow, as has been shown by many scientists, the pulsation of stream velocity bears the character of a stationary random process. The accuracy of the determination of mean velocity $\overline{V}$ depends on the duration of its recording T. The standard current meter exposure $T = 100$ sec, corresponds to the following errors:

velocity at the point close to the surface of water	2 per cent
mean velocity of the vertical:	
at 5 point measurements:	1.2 per cent
at 2 point measurements:	2.0 per cent

The correlation function is one of the characteristics of the random process of the stream velocity pulsation. Its shape (the positive values of T only are considered) is established as a result of field investigations of the turbulent structure of streams:

$$R_v(T) = e^{-\alpha T} \tag{7}$$

where α ranges from 0.1 to 0.3 l/sec.

The errors of velocity measurements at a point σ, its mean square value σ_v and the duration of observation T are related by an approximate dependence

$$\sigma \approx \sigma_v \cdot \sqrt{\frac{2}{\alpha T}}, \tag{8}$$

if $\alpha T \gg 1$.

If velocity is measured over different current meter exposure times T_1 and T_2, then the ratio of corresponding errors will be:

$$\frac{\sigma_1}{\sigma_2} = \sqrt{\frac{T_2}{T_1}}. \tag{9}$$

From equation 9 it follows that a relatively small increase of error during velocity measurements may allow a considerable acceleration of measurements; in this case, equation 6 shows that the random error of discharge measurements will increase only slightly. If it equals, say, 10 per cent, then a tolerable increase of the error of mean velocity measurements on the vertical for the two-point method might be from 2 to 3 per cent. The same increase in error may be also accepted in the measurements of velocity at individual points; according to formula 9 it will result in the reduction of current meter exposure time to 40 or 45 sec instead of the accepted time of 100 sec.

Computation by formula 6 shows that, in the majority of cases, it is expedient to reduce the duration of velocity measurements at a point by three or four times, i.e. to not more than 30 sec. If the current is slow, the time of current meter exposure at a point is limited by the interval between two signals.

During ice cover the error of discharge measurements may be considerable due to pulsation. But even in this case, it is quite possible to reduce considerably the duration of point measurements without detriment to the accuracy of the result. Experimental investigations on the Svir river show that the error caused by pulsation on the verticals and on the discharge as a whole is not much increased if the duration of current meter exposure is considerably reduced (from 200 to 30 sec).

To verify the effect of reduction of current meter exposure on the accuracy of discharge measurements, a series of experimental discharge measurements on the Neman river and on a large irrigation canal has been made. The discharge, measured in the usual way, i.e. by the detailed method, is accepted as the reference discharge. Mean square difference between the reference discharge and the discharge measured with reduced times of current meter exposure is equal to 1.2 per cent with the five-point method and 1.6 per cent with the two-point method using 118 measurements.

The results obtained justify a considerable reduction (down to 30 sec) in duration of flow velocity measurements at a point if the streamflow is estimated by the discharge curve, which reduces the effect of random errors during measurements; this reduction is expedient for measurement of velocity at 5 and at 2 points if the number of verticals is not less than 5.

In winter, under ice conditions, when streamflow is computed by interpolation between measured discharges or by transition coefficients, the random error considerably affects the result and reduction in time of current meter exposure cannot be recommended.

The accelerated method of discharge measurement should not be applied during a rating period and in cases when the averaged velocity is of special interest; in this case the current meter exposure may be 5-10 and even 20 min, depending on the required accuracy of measurements.

The effect of averaging in space and time to accelerate measurements is best used with the integration methods of discharge measurements. It is expedient to apply the method of flow velocity integration down the vertical as the basis for the rationalization of discharge measurements. This method does not require complex boat or cable-car equipment to transfer the current meter in the stream. The same equipment

(wading rod and cable) as for point measurements is used. The only additional equipment consists of the automatic recorder of time and of number of revolutions of the current meter (electric pulse counter). There is no need to design special regulators for uniform lowering of the current meter: it is possible to do it manually using metronome signals. Experimental discharge measurements by the velocity integration method on the Volga, Neva and Amu-Darya gave results approximately the same as those obtained with the two-point method with a considerable reduction in measurement time.

A considerable saving in time (two or three times) can be achieved by taking discharge measurements from cable-cars, cable ways and telphers which facilitate a rapid change in instrumental position. The latter may be obtained on large rivers from an unanchored boat, though the technique of such measurements needs special investigation.

Ultrasonic measurement of discharge in rivers

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Abstract. Progress in improvement of ultrasonic flowmeters in Japan since 1964 is reported. Data obtained by ultrasonic flowmeters are compared with those from rotating current meters; agreement is very good. Some limitations were experienced during field tests. If these limitations are resolved, the author believes that it will be a most useful instrument for flow measurement.

MESURE PAR ULTRASONS DES DEBITS DES RIVIERES

Résumé. Depuis 1964, l'élaboration d'un système de mesure des débits à l'aide des ultrasons à été poursuivie au Japon. L'amélioration apportée récemment à ce système est décrite dans cette communication. Les données obtenues par cette méthode à ultrasons sont comparables à celles obtenues par jaugeage au moulinet. Quelques limites de l'application du système ont été décelées pendant les essais sur place. L'auteur pense qu'il sera utile pour l'hydrométrie de réduire ces limites.

MEDICIÓN ULTRASÓNICA DE CAUDAL EN RÍOS

Resumen. El medidor de caudal ultrasónico ha sido mejorado desde 1964 en el Japón. En la presente comunicación se da cuenta de la mejora actual. Se comparan datos obtenidos por medidores de caudal ultrasónicos con los obtenidos por medidores de corriente rotativos y se observa una satisfactoria concordancia. En los ensayos de campo se han experimentado algunas limitaciones; el intento de reducir dichas limitaciones, a juicio del autor, sería de la máxima utilidad para la hidrometría.

ИЗМЕРЕНИЕ РАСХОДА ВОДЫ В РЕКАХ С ПОМОЩЬЮ УЛЬТРАЗВУКА

Резюме. В Японии ультразвуковой расходомер постепенно улучшался с 1964 года. Достигнутое к настоящему времени усовершенствование прибора изложено в докладе. Данные, полученные ультразвуковым расходомером, сопоставлены с таковыми, измеренными вертушкой. Их совпадение весьма хорошее. Шекоторые ограничения были выявлены опытным путем в процессе полевых испытаний. Пытаясь уменьшить эти ограничения, автор надеется, что прибор со временем будет весьма полезным в гидрометрии.

INTRODUCTION

As flow measurement in rivers is very important for flood control and water resources development, various improvements in measurement have been made by many hydrologists over a long period of time. Nevertheless, discharge measurement still remains troublesome and difficult. The ultrasonic method is one of the most promising techniques to have been developed in recent years. In this paper, the author reports the results of field tests and some limitations of this method.

Firstly, some merits of the ultrasonic method of discharge measurement are:

1. Automatic measurement saves labour;
2. Continous measurement can follow up a sudden change of discharge;
3. Electronic measurement permits telemetering and digital processing of data;
4. This measurement does not influence the flow;
5. This measurement is suitable for tidal flow or for slow flow because it is independent of head of water;
6. Ultrasonic waves propagate through water, so the mean velocity can be obtained.

There are various kinds of ultrasonic systems for measuring flows: (a) beam deflection system; (b) phase comparison system; (c) pulse transit system; (d) sing around system; and (e) Doppler system.

The sing around system has no relationship to ultrasonic velocity. The pulse transit system is effective for measurement of velocity components in a certain direction. The Doppler system permits measurement of both the flow velocity and the concentration of suspended material.

PRINCIPLE OF ULTRASONIC SYSTEMS

As is well known, a discharge Q is defined as

$$Q = \int_A v(y, z)\, dy\, dz \tag{1}$$

where y is the lateral axis, z is the vertical axis, v is the current velocity at (y, z) and A is the whole cross-section, as shown in Figure 1.

The integration can be made on each axis.

$$V(z) = \int_B v(y, z)\, dy \tag{2}$$

$$V'(y) = \int_H v(y, z)\, dz. \tag{3}$$

So Q is, of course, obtained by integrating V or V'

$$Q = \int_H V(z)\, dz = \int_B V'(y)\, dy \tag{4}$$

$V(y)$ is called "discharge per unit width" at y.
V/B is the lateral mean velocity at Z, where B is the width.

Since the vertical distribution of velocity has been discussed from the viewpoint of hydraulics, its pattern and the correction factor for obtaining V' by means of point velocity measurement are already determined. However, the lateral distribution of velocity is not yet known. Although the Cameron effect in aerophotogrammetry brings to light the very complicated structure of velocity on the river surface, the hydraulic law governing this structure has not been made clear.

An ultrasonic method exists which averages the complicated pattern of velocity distribution method.

Ultrasonic transducers are set horizontally on both sides of the river and the ultrasound travels laterally in the flow. The set of the ultrasonic flowmeter is shown in Figure 2. T_1 and T_2 are a pair of transducers. First, T_1 emits an ultrasonic wave

packet in response to an electrical signal. This wave packet goes through the water to T_2. The travel time from T_1 to T_2 is given by

$$t_1 = L/(c + v \cdot \cos\theta) \tag{5}$$

where L is the distance, c is the ultrasonic velocity in still water, v in the flow velocity and θ is the angle between the flow direction and the ultrasonic beam. Second, T_2 emits an ultrasonic wave packet and T_1 receives it. The travel time from T_2 to T_1 is given by

$$t_2 = L/(c - v \cdot \cos\theta). \tag{6}$$

The pulse transit system derives v from the difference between t_1 and t_2.

$$\Delta t = t_2 - t_1 = \frac{2vL \,.\, \cos\theta}{c^2 - v^2 \cos^2\theta} = \frac{2vL \,.\, \cos\theta}{c^2} \tag{7}$$

where $v^2 \cos^2\theta$ is much smaller than c^2. Expression 7 means v is proportional to the difference of travel time Δt, so v is easily obtained. Attention must be paid to the ultrasonic velocity c, which is about 1,400 m/sec in normal situations and changes with water temperature, density and so on. We must correct the errors due to change of c.

This system was improved. Three transducers are set in the river as shown in Figure 3. Transducer T_2' plays the role of a reflector. t_1 is the travel time from T_1 via T_2 to T_3.

$$t_1 = \frac{L}{c + v \cdot \cos\theta} + \frac{L'}{c + v \cdot \cos\theta'} \tag{8}$$

$$t_2 = \frac{L}{c - v \cdot \cos\theta} + \frac{L'}{c - v \cdot \cos\theta'} \tag{9}$$

where L, L', θ and θ' are as shown in Figure 3.

$$\Delta t = t_2 - t_1 = \frac{2 \cdot L \cdot v \cdot \cos\theta}{c^2 - v^2 \cos^2\theta} + \frac{2 \cdot L' \cdot v \cdot \cos\theta'}{c^2 - v^2 \cos^2\theta'} = \frac{2v}{c^2}(L \cdot \cos\theta + L' \cos\theta'). \tag{10}$$

If α is the angle between $\overline{T_1 T_3}$ and the stream direction, $(L \cos\theta + L' \cos\theta')$ equals $d \cdot \cos\alpha$. Thus we obtain the expression

$$\Delta t = \frac{2 \cdot v}{c^2} \cdot d \cdot \cos\alpha = \frac{2d}{c^2} \cdot v \cdot \cos\alpha. \tag{11}$$

The parallel component of velocity to $\overline{T_1 T_3}$ is derived from expression 11; consequently the discharge becomes known.

The results of this system are independent of meandering of flow in a river channel. Furthermore, a cable across the river is unnecessary.

The sing around system is also simple. In Figure 2, an electrical signal is induced at T_2 at the same time as the wave packet arrives at T_2. Then the circuit works and T_1 emits a new wave packet again. Thus a signal is running around T_1, T_2 and an electric conduit; this leads to the name "sing around system".

The frequency of singing around is defined as "how many times the signal sings around through this system in a unit of time"

$$f_1 = \frac{1}{t_1} = \frac{c + v \cdot \cos\theta}{L}. \tag{12}$$

Next T_2 emits an ultrasonic wave packet. It goes through water upstream and arrives at T_1.

The frequency of singing around from T_2 to T_1 is

$$f_2 = \frac{1}{t_2} = \frac{c - v \cdot \cos\theta}{L}. \tag{13}$$

The difference between f_1 and f_2 is determined

$$\Delta f = f_1 - f_2 = \frac{2v \cos\theta}{L}. \tag{14}$$

Δf is proportional to the flow velocity v.

If L and θ are fixed, v is easily derived from only Δf. The result is not affected by the ultrasonic velocity c but an error in θ cannot be ignored. In a big river the measuring period is long, which may lead to error due to fluctuation of flow pattern. A cable across the river is necessary.

PRELIMINARY EXPERIMENT

Before setting up the ultrasonic flowmeter in natural rivers, a preliminary experiment was carried out in August 1964 in the channel for hydraulic experiments at the Public Works Research Institute. The dimensions of this channel are as follows: width, 1.8 m; channel depth, 1 m; maximum discharge, 1.5 m^3/sec.

Transducers were fixed to the inside walls of the channel as shown in Figure 4. They were made of ceramics which resonated at 400 kHz.

According to the data in Figure 4

$$\Delta f = 0.627\, v$$

where v is in m/sec. The discharge Q(m^3/sec) is

$$Q = 1.78hv$$

where h is the water depth in metres.

A comparison of Q with Q_N (the discharge measured by the rectangular notch) is shown in Figure 5. The difference between Q and Q_N is less than 5 per cent.

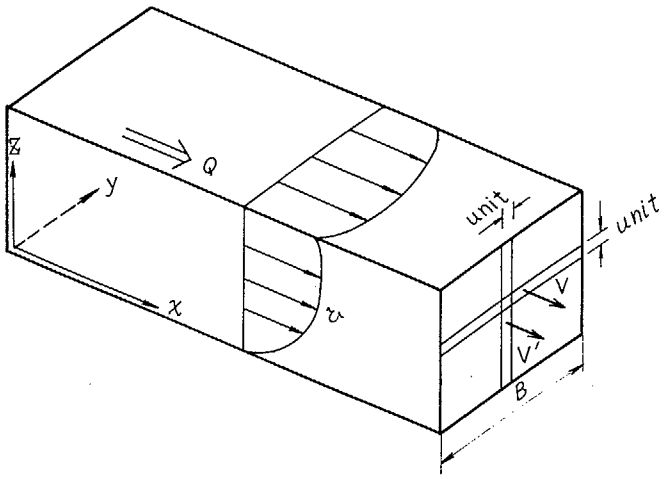


Figure 1. *Schematic model of open channel flow*

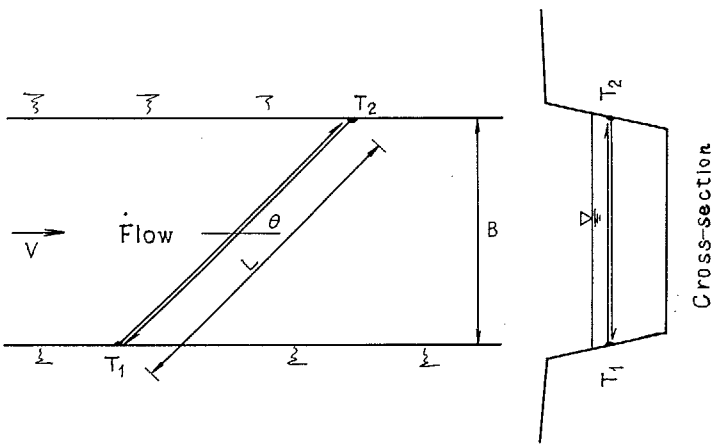


Figure 2. *Installation of transducers*

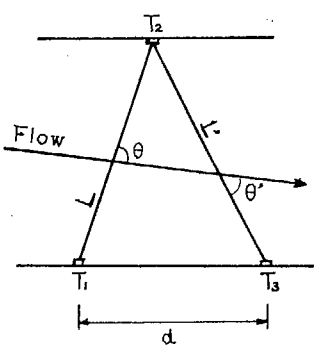


Figure 3. *Three transducers set in the river*

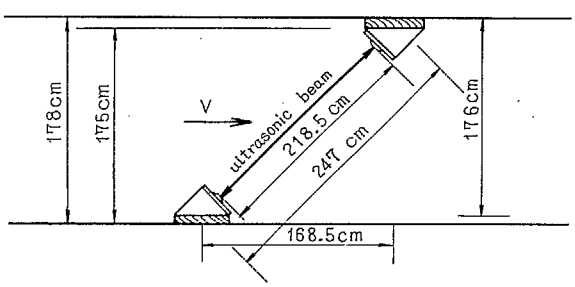


Figure 4. *Set-up of preliminary experiment*

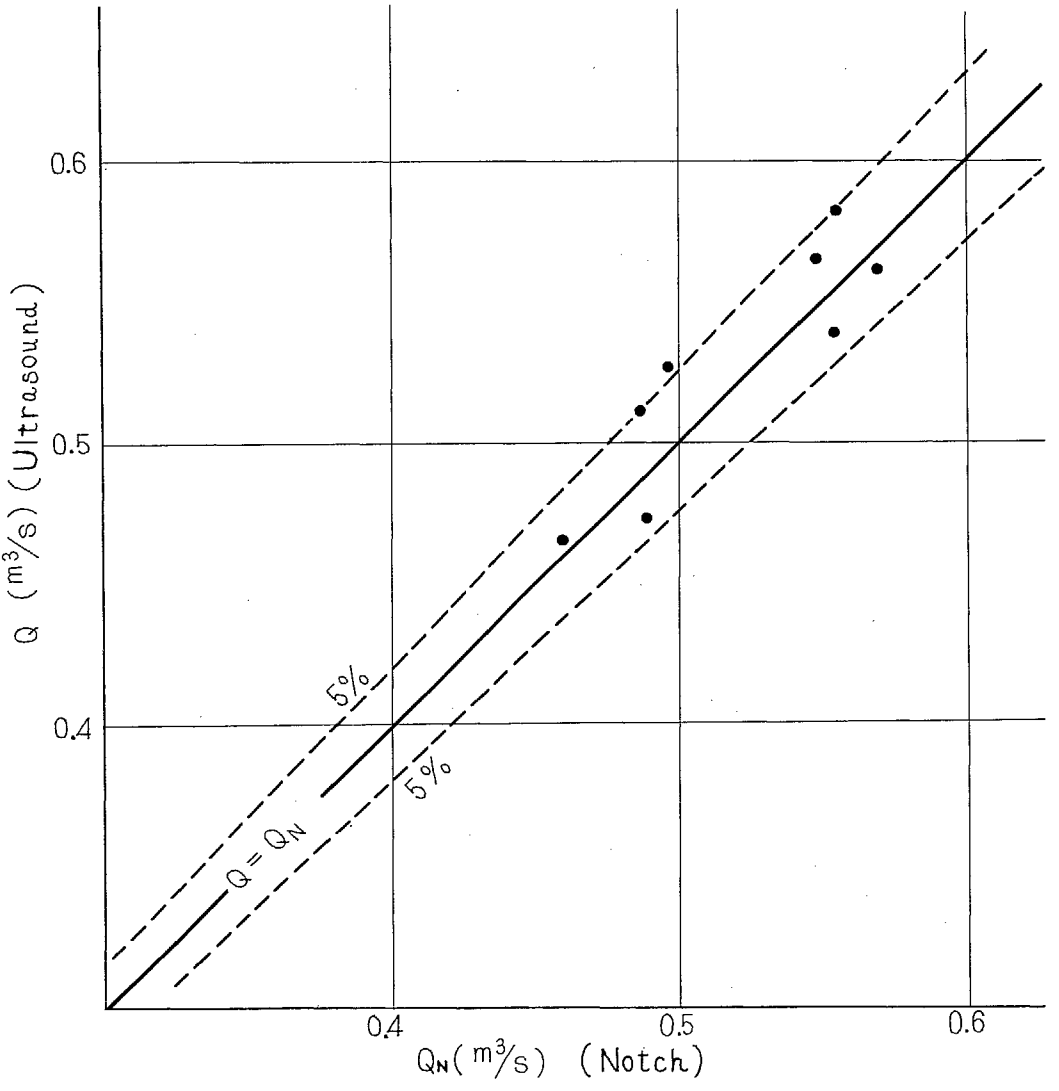


Figure 5. *Comparison of discharges at the preliminary experimental stage*

FIELD TESTS

After the success of this preliminary experiment, ultrasonics were applied to flow measurement in the field.

In Japan, river beds slope very steeply and streams are very rapid. The climate is severe and floods occur unexpectedly. During floods, substantial quantities of suspended material are transported in the water. It is inconvenient to test the ultrasonic flow meter in rivers only during floods.

Firstly, it was used for tidal rivers, where the stream is not so rapid and there is less suspended material. Two sites were selected for test. One was Itikawa-basi in River Edo; the other was Yasiro, the outlet of the Kasumigaura lake (Fig. 6). The sing around system was tested at Itikawa-basi, and the pulse transit system at Yasiro.

The dimensions of the rivers and ultrasonic facilities are as follows:

Itikawa-basi (sing around system)

channel width	120 m
depth	4 m
tidal amplitude	1 m
ultrasound	200 kHz
sing around period	40.96 sec
diameter of transducer	8 cm
angle between the stream direction and the ultrasonic beam	42° 22′ 20″

Yasiro (pulse transit system)

channel width	235 m
depth	4 m
tidal amplitude	30 cm
ultrasound	100 kHz
diameter of transducer	5 cm
distance between T_1 and T_3	50 m

The transducer was fixed to piles set in the river bed (Fig. 7).

During the period of the operating test of the ultrasonic flowmeter, stream velocity was also measured by rotating current meters, such as the Price meter and the Ekman-merz current meter.

Figures 8 and 9 show a comparison of ultrasonic mean velocity with rotating current meters' mean velocity. The 45° line in Figure 8 means exact coincidence. Although there is some scatter about this line, there is general agreement. Figure 9 shows the time changes of current velocity using the Price meter, the automatic recording current meter (rotating), and the ultrasonic flowmeter. The value obtained from the ultrasonic flowmeter agrees well with that from the Price meter, though the latter is insensitive at low velocity. This encourages the use of the ultrasonic flowmeter instead of a rotating current meter in a river.

SOME LIMITATIONS TO APPLICATION

During the field tests limitations were discovered, and will now be discussed.

Width of the channel. Because of geometric attenuation, a strong ultrasonic pulse is necessary to measure the flow in a wide channel. If a strong pulse can be emitted, the

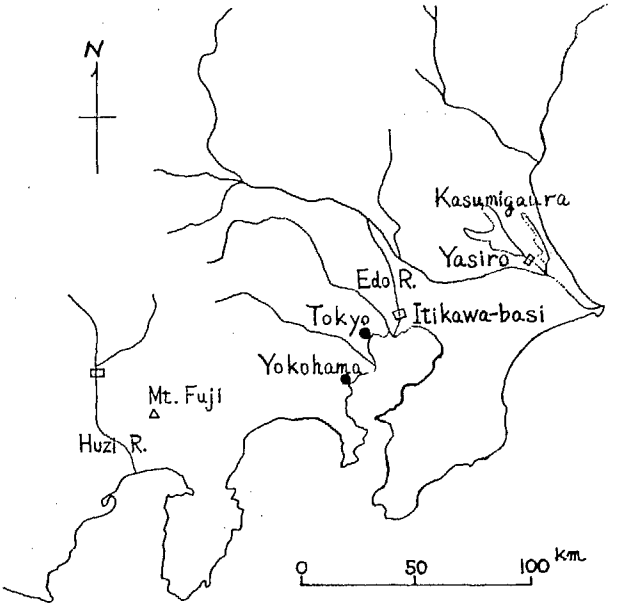


Figure 6. *Location of field tests (indicated by □)*

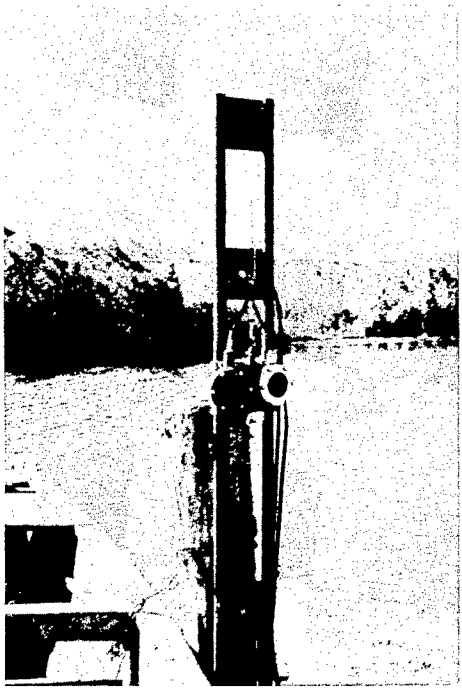


Figure 7. *Transducers fixed to a steel pile*

ultrasound reflected at the surface or the bottom interferes with the direct ultrasound in a shallow channel. The ultrasound must propagate along the path at some distance from the surface. This distance is determined by the channel width and the temperature gradient near the surface. Roughly speaking, in a normal situation, this distance must be greater than 50 cm.

The travel time of the ultrasonic pulse from one side of the channel to the other is proportional to the channel width. Suppose the ultrasonic velocity is 1,400 m/sec and the angle between the channel direction and the ultrasonic path is 45°. In a channel of 100 m width, it takes 0.1 sec. for an ultrasonic pulse to travel from one side to the other; 100 times of "singing around" of the pulse is counted in 10 sec. This increases the accuracy of measurement by the sing around system, which takes time even if a frequency multiplier is used. The pulse transit system is convenient for measuring flow in a wide channel because the difference between the transit time of an upstream pulse and that of a downstream pulse increases with the width of the channel.

Channel form. According to asymmetry, winding and changing of the cross-sectional form of the channel, the direction of the flow is sometimes unstable. For hydrological purposes, the discharge through a certain section should be measured. A system in which deflection of the flow causes error is fatal. The improved pulse transit system (Fig. 3) is useful because the component of the velocity parallel to $\overline{T_1\ T_3}$ can be obtained.

Some consideration should be given to reverberation in a concrete channel, since this induces meaningless pulses which cause errors in measurement.

Sediment and bubbles. When the flow velocity is high during floods, sediment contents are great. When water molecules and sediment particles collide, they generate noise. When bubbles burst, they also generate noise. A pulsative noise or a big noise, the amplitude of which is bigger than the trigger level of the ultrasonic detector, is taken for a signal by this detector.

Attenuation caused by sediment or bubbles is expressed as an exponential function e^{-kx} where k is a coefficient depending upon the concentration of sediment or bubbles, and x is the distance between the transmitter and receiver. Geometric attenuation varies inversely as the distance. So e^{-kx} tends to zero more rapidly, i.e. attenuation caused by sediment and bubbles is more serious than geometric attenuation.

To distinguish signal from noise, high frequency ultrasound is better; to reduce attenuation caused by sediment and bubbles, low frequency ultrasound is better. Taking these facts into consideration, ultrasound of 100-200 kHz is selected as the best frequency.

Flow velocity. In a low velocity flow, the pulse form must be sharp and clear. If it is distorted, the measured value of the velocity will be relatively inaccurate. Thus ultrasonic systems using the pulse are inadequate. The phase comparison system is better for measuring low velocity flow.

Temperature. The velocity of ultrasound in water is a function of water temperature, salinity and density. In the formulae for obtaining flow velocity by the pulse transit system and the phase comparison system, the velocity of ultrasound should be

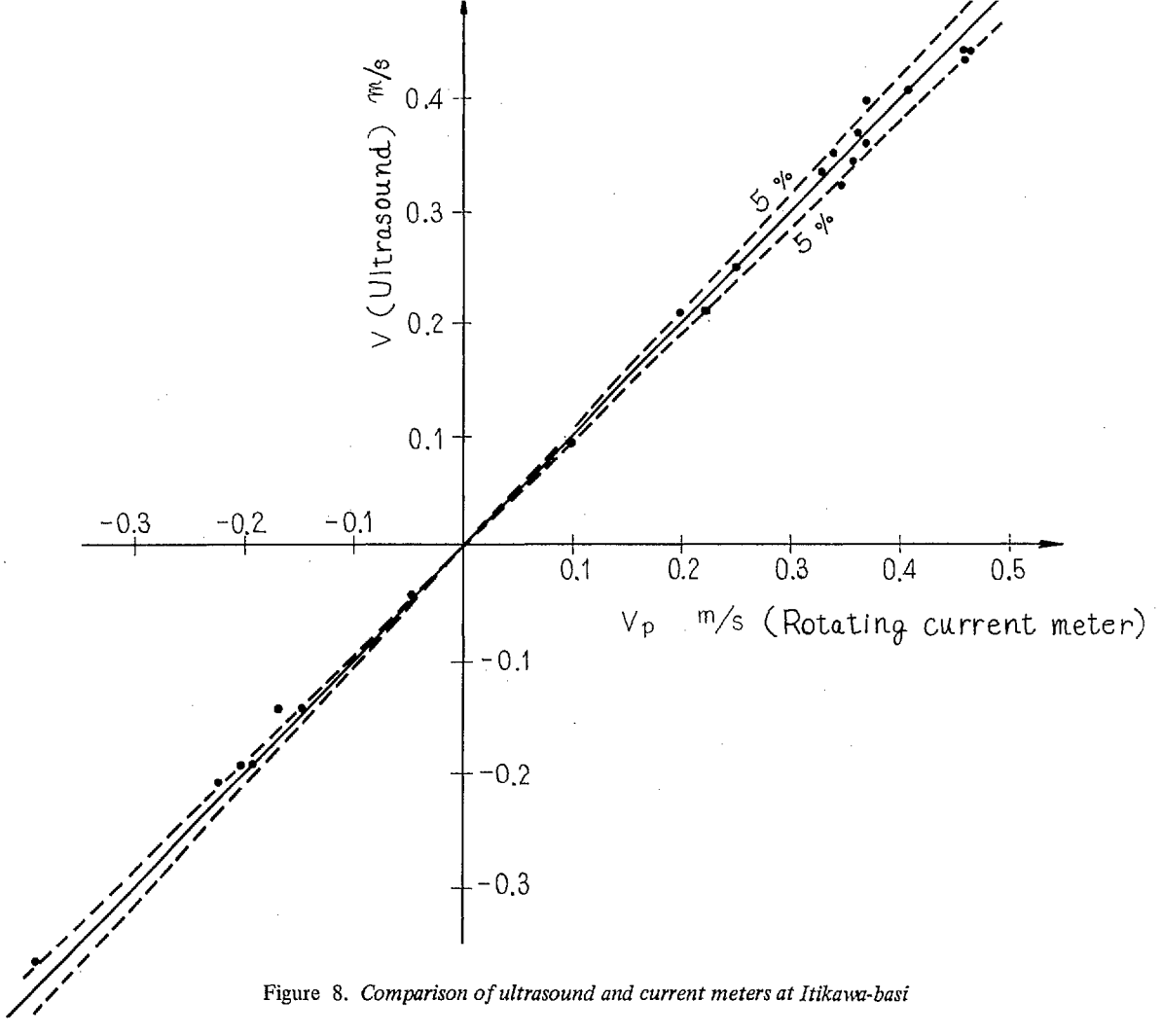


Figure 8. *Comparison of ultrasound and current meters at Itikawa-basi*

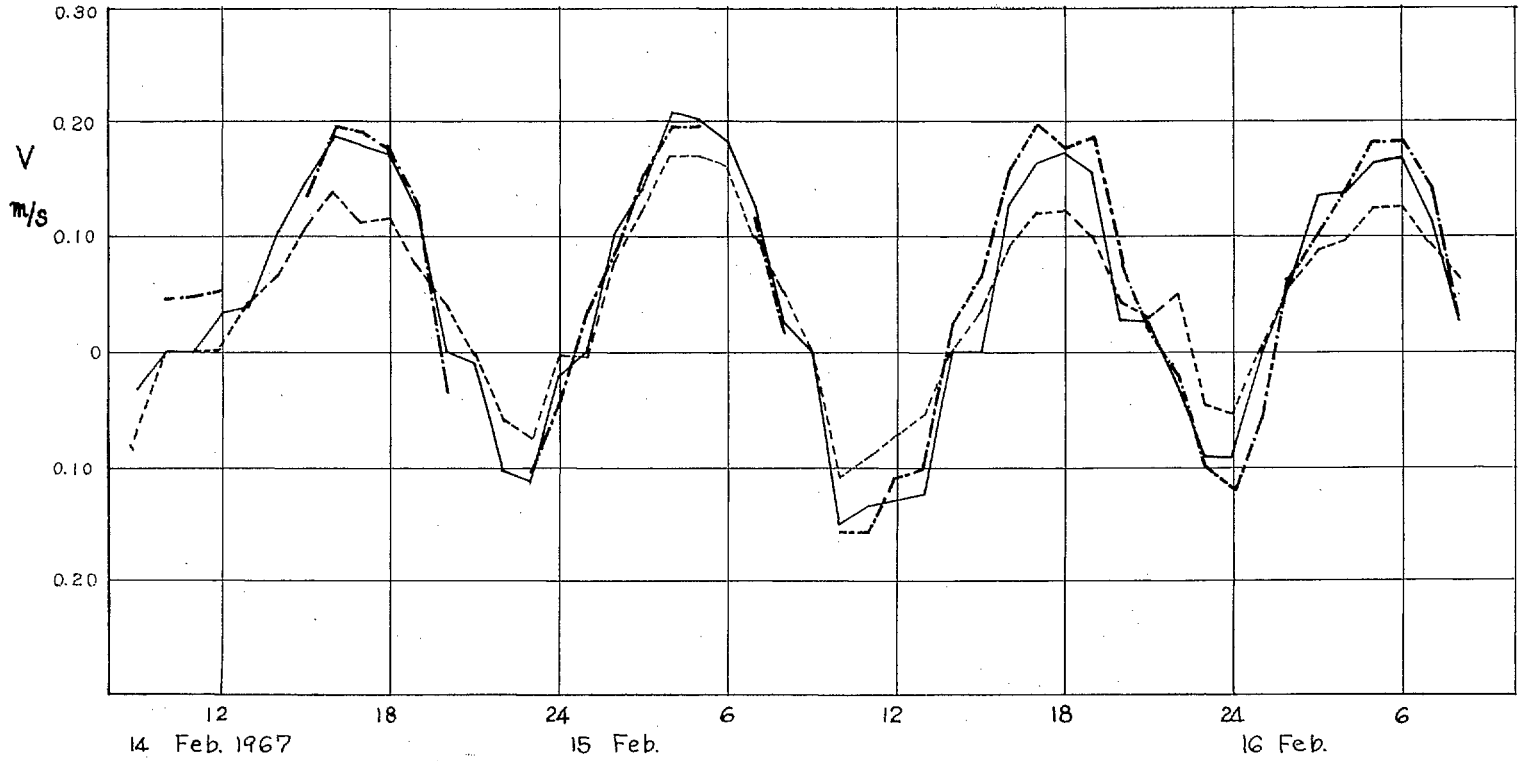


Figure 9. *Comparison of Price meter automatic recording current meter and ultrasonic flow-meter*

known. Therefore, water temperature should be observed in order to correct the flow velocity. In the sing around system the flow velocity is independent of the velocity of ultrasound.

Temperature gradient. Even if the effect of temperature is completely eliminated, the vertical gradient of water temperature seriously affects the measurement of discharge.

When the system, set at the outlet of the Kasumigaura, was out of order during the warmest hours in summer, a working group was organized and much time and labour have been devoted to an investigation. Teramoto and his collaborators surveyed the temperature distribution in the cross-section and found considerable temperature gradients in the water. Okushima calculated the deflection of the ultrasonic beam. It was concluded that the ultrasonic system broke down because of deflection of the ultrasonic beam when the temperature gradient near the surface of the river was marked. This kind of breakdown has not yet been found at the outlet of another lake in a similar situation, but did occur in one river during a snow storm.

CONCLUSIONS

The most significant merit of the ultrasonic flowmeter is that the lateral mean velocity can be obtained.

Values for the mean velocity obtained in various kinds of rivers (rapid, tidal, an outlet of a lake, etc.) by the ultrasonic system agreed well with those obtained by Price meters.

During field tests, some limitations were found which are very important for future development of ultrasonic flowmeters. Some of them have already been overcome, but others have not. The author believes that the ultrasonic flowmeter will prove to be the best way to measure the discharge of rivers in any condition.

Measurement of discharge as inflow into leaky reservoirs

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Abstract. Surface runoff is measured as into floodwater detention reservoirs in the Edwards Plateau near Sonora, Texas, United States. The Edwards Plateau is a karst area in south-west Texas and the reservoirs are underlain by cavernous limestone. Research is being conducted by the Agricultural Research Service of the United States Department of Agriculture to determine the disposition of water impounded in five leaky reservoirs.

Losses from detention storage occur at variable rates throughout times of inflow and outflow. Estimates of loss rates are necessary to accurately determine the total volumes of inflow.

The installations are described, and the procedure for estimating reservoir loss rates, duration of inflow and total inflow volume is illustrated. Methods of estimating loss rates from continuous reservoir stage records and mathematical expressions are presented.

MESURES DU DEBIT S'ECOULANT DANS DES RESERVOIRS PRESENTANT DES FUITES

Résumé. L'écoulement de surface est mesuré dès qu'il pénètre dans les réservoirs de stockage du plateau d'Edwards, près de Sonora, Texas (Etats-Unis). Le plateau d'Edwards, région karstique, est situé dans le sud-ouest du Texas et les réservoirs sont situés dans des roches calcaires caverneuses. Des recherches sont faites et dirigées par le Service de recherches agricoles des Etats-Unis, Département de l'agriculture, pour déterminer la distribution de l'eau accumulée dans les cinq réservoirs qui fuient.

Les pertes d'eau stockée dans les réservoirs varient en fonction des temps d'entrée et de sortie. Il est nécessaire de faire une estimation des pertes pour pouvoir déterminer exactement le volume total des eaux qui pénètrent. Une description des installations est fournie ainsi que le procédé pour estimer la perte des réservoirs, la durée de l'entrée des eaux et le volume total entrant. Les méthodes pour estimer les pertes des réservoirs sont obtenues par des enregistrements continus et les expressions mathématiques sont explicitées.

MEDICIONES DE CAUDAL AL ENTRAR EN EMBALSES CON FUGAS

Resumen. La escorrentía superficial se mide desde que entra en los embalses de detención en la Edwards Plateau, cerca de Sonora, Texas. La Edwards Plateau es una zona kárstica al sudoeste de Texas, y los embalses se encuentran en calizas. El Servicio de Investigaciones Agronómicas del U.S. Department of Agriculture está realizando una investigación para determinar los volúmenes almacenados en cinco embalses con fugas.

Las pérdidas de almacenamiento se presentan con valores variables en los momentos de flujo entrante y saliente. Es necesario proceder a la estimación de las pérdidas para determinar con precisión el volumen total de aportación.

Se describen las instalaciones indicándose el procedimiento utilizado para estimar los valores de pérdida en el embalse, la duración del flujo de entrada y el volumen total de este último y se desarrolla el método para la estimación de las pérdidas a partir de los registros continuos de nivel de embalse, dándose las expresiones matemáticas correspondientes.

ИЗМЕРЕНИЕ РАСХОДОВ ВОДЫ ПО ПРИТОКУ В ПРОТОЧНЫЕ ВОДОХРАНИЛИЩА

Резюме. Поверхностный сток измерялся по притоку в водохранилища, созданные для задезжания паводочных вод на плато Эдвардса близ Сонора, штат Техас. Плато Эдвардса представляет собой карстовую область на юго-западе Техаса; водохранилища выстлани пещеристыми известняками. Исследования проводились сельскохозяйственной исследовательской службой Министерства сельского хозяйства США с целью определениязапаса воды, накопленной в пяти проточных водохранилищах.

Потери из накопленного объема воды происходят с различной интенсивностью на протязении всех периодов притока и оттока. Оценка величин потерь являлась необходимой для точного определения полных объемов притока.

Описаны установки и иллюстрирована методика для оценки величин потерь из водохранилища в период притока и полного объема притока. Предложены способы оценки потерь на основании непрерывных записей уровней в водохранилище и даны математические выражения.

INTRODUCTION

The United States Department of Agriculture's Agricultural Research Service in co-operation with the Soil Conservation Service and the Texas Agricultural Experiment Station is conducting hydrologic studies in the Edwards Plateau of Texas, U.S.A. (Knisel, 1965). Research was begun after completion of five flood-detention reservoirs in the 48-square-mile Lowrey Draw watershed near Sonora, Texas. The objective of the study is to determine the disposition of runoff water impounded in flood-detention reservoirs in the karstic Edwards Plateau.

The Edwards Plateau is underlain by limestones that typically contain interconnected fissures, solution channels, and caverns. The chance occurrence of variable size openings at or near the bottom of reservoirs results in variable loss rates among the reservoirs.

In order to attain the objective, it is necessary to determine the amount of inflow into the reservoirs. Determination of inflow requires the measurement of reservoir stage and outflow, and the estimation of losses. This paper describes the methods of estimating losses, time of inflow and total inflow.

HYDROLOGIC INSTALLATIONS

The flood-detention reservoirs in Lowrey Draw are typical of those constructed by the Soil Conservation Service in their upstream-flood control work. A schematic drawing of a typical structure is shown in Figure 1. The reservoir is formed by an earthen dam and overflows through two spillways. The lower, or principal, spillway consists of a concrete drop inlet with antivortex deck and concrete pipe beneath the dam. The higher (emergency) spillway is an open channel seeded to grass or excavated in rock, as in the Lowrey Draw reservoirs.

Detailed topographic surveys were made of the sediment and detention pools of the five reservoirs in Lowrey Draw watershed. Stage-storage relationships for each reservoir were determined from these surveys.

Head-discharge relationships for the principal spillways were calculated. These calculations were based on hydraulic laws, on the dimensions and elevations of the

structure, and on the frictional resistance of the conduit. Estimates of friction factors for the conduits were based on photographs of surfaces of pipe of known friction factor (Straub *et al.*, 1960).

The bubbler-type gauges, used extensively by the U.S. Geological Survey, and described by Barron (1960) and strip-chart recorders were installed to provide continuous records of reservoir stage.

RESERVOIR INFLOW AND LOSS CONDITIONS

Floodwater-detention reservoirs like the one shown in Figure 1 have three possible inflow conditions: (a) inflow with maximum stage below principal spill-way elevation (no outflow); (b) inflow with maximum stage above principal spillway elevation, but below emergency spillway elevation (principal spillway outflow); and (c) inflow with maximum stage above the emergency spillway elevation (outflow from principal and emergency spillways). Each condition may or may not have losses superimposed. Accurate estimates of total inflow from reservoir volume change must be adjusted for spillway discharge that occurred during inflow and any losses that occurred during inflow. Reservoir stage records affected by outflow and losses make difficult the determination of the time that inflow ends, and consequently, complicate the determination of the total volume of inflow.

INFLOW AND LOSS COMPUTATIONS

Inflow conditions 1 and 2 have been observed at the reservoirs in Lowrey Draw watershed. Two examples of each condition have been selected for this presentation. One example of each condition represents low loss rates (reservoir S-12) and one represents high loss rates (reservoir S-10). The examples will demonstrate the difficulty and variability encountered in reservoir-inflow determination.

Records of inflow condition 1 for reservoirs S-10 and S-12 were examined. The hydrographs for the events meeting this condition are shown in Figures 2 and 3, respectively. The loss rates are quite different for the two reservoirs, being 0.150 ft per hr for S-10 and 0.018 ft per hr for S-12. However, the loss rates in feet per hour were found to be relatively constant for each reservoir irrespective of stage. This constancy of loss rate simplified the problem of determining total volume of inflow.

Extending the average stage loss-rate slope backwards in time to the point where the actual curve deviated from the extended line determined the time at which inflow ended. Minor variations in loss rate tend to make the determination inexact.

The recorded beginning of inflow and the estimated end of inflow define the duration within which loss volume must be estimated. The estimated loss must be added to the volume in storage at the end of inflow to determine the total inflow.

The total inflow volume is estimated by determining the loss volumes for small time increments. The smaller the time increment chosen, the more accurate the estimate will be. The least time division of the recorder chart is the most convenient increment to use. In the case of this study, the least time division is 15 min. and its use yielded statisfactory accuracy. The sum of the estimated loss volume added to the volume of water in the reservoir at the end of the summation period is the total inflow up to that time. This statement can be expressed mathematically by the equation:

$$I_t = S_t + \sum_{i=0}^{i=t} L_i \qquad (1)$$

where I_t = total inflow at time t; S_t = volume of storage in the reservoir at time t; and L_1 = volume of loss during the ith increment.

As in the example for reservoir S-10, Figure 2, 15 min. after inflow began the stage was 0.493 ft. If losses had not occurred, the estimated stage would have been 0.493 ft plus 1/4 of the 0.150 ft per hr stage loss rate previously determined, or 0.530 ft. The volume in storage at the recorded stage is 0.0092 acre-ft and at the estimated stage is 0.0138 acre-ft. The difference between the two, 0.0046 acre-ft, is the loss for the interval. At the end of the second 15-min. interval, the stage was 0.506 ft. If losses had not occurred during the interval, the estimated stage would have been 0.544 ft. Corresponding volumes for the actual and estimated stages are 0.0099 and 0.0131 acre-ft, respectively, or a loss of 0.0032 acre-ft. The accumulated loss from the beginning of inflow is 0.0078 acre-ft. Continuing this procedure throughout the inflow gives an indicated inflow (from maximum reservoir stage) of 69.58 acre-ft, and accumulated losses of 10.50 acre-ft. Thus, the total inflow was the sum of the two, or 79.63 acre-ft, which is 14.44 per cent more than the inflow indicated by actual reservoir stage.

For inflow condition 2, with the principal spillway functioning, the procedure is not as simple. The change in stage due to outflow is not a straight line and the extension of a line backwards in time, as was done for condition 1, will not accurately define the time that inflow ends. The procedure is one of trial and error. First, the stage loss rate must be estimated from a portion of the record after principal spillway flow ends. A point in time is then selected on the stage record when it is thought that inflow has ended. For an increment of time following the selected point, the volume of outflow is determined from the head-discharge relationship for the spillway and the heads at the beginning and end of the time interval determined from the reservoir stage record.

The volume of loss during the interval is estimated as in the previous example. The sum of the outflow and loss is compared with the change in volume for the interval indicated by the stage record. If the indicated volume change is less than the sum of outflow and losses, then inflow is indicated for the interval. This trial-and-error balance procedure must be continued for later intervals until the two volume changes are equal. Once the end of inflow is determined, the previously described procedure is used to estimate the total losses from the beginning of inflow. The incremental procedure is the same to the point where principal spillway flow begins. After that time, the spillway head at the beginning and end of the increment and, in turn, the corresponding discharge rates and volume for the increment are determined. Estimation of stage at the end of the increment to compensate for losses is made after spillway discharge is determined. Actual stages are used for spillway discharge. Total inflow to the end of the increment is indicated storage plus accumulated losses plus accumulated spillway discharge. The equation expressing this relationship is:

$$I_t = S_t + \sum_{i=0}^{i=t} L_i + \sum_{i=0}^{i=t} (Qp)_i \qquad (2)$$

where $(Qp)_i$ is the volume of principal spillway discharge tor the ith interval.

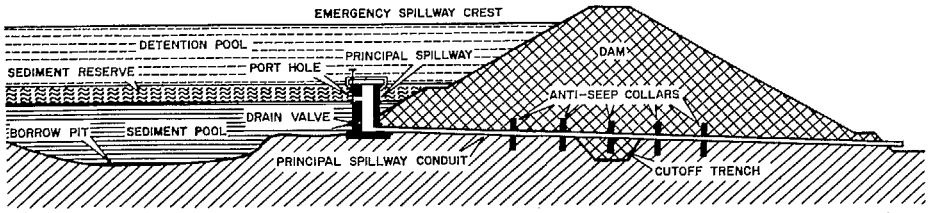


Figure 1. *Section of a typical floodwater retarding structure*

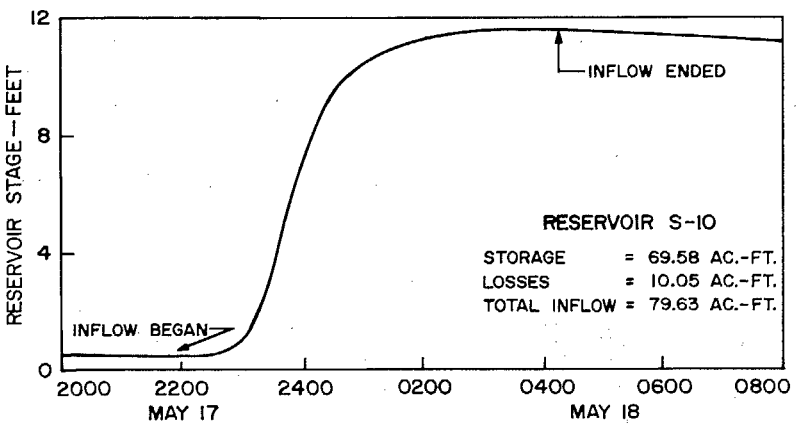


Figure 2. *Time-stage record for reservoir S-10, 17/18 May 1965*

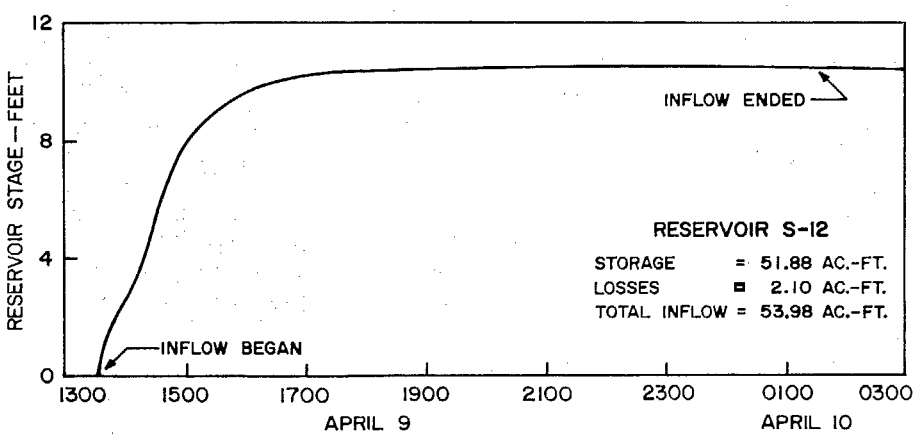


Figure 3. *Time-stage record for reservoir S-12, 9/10 April 1968*

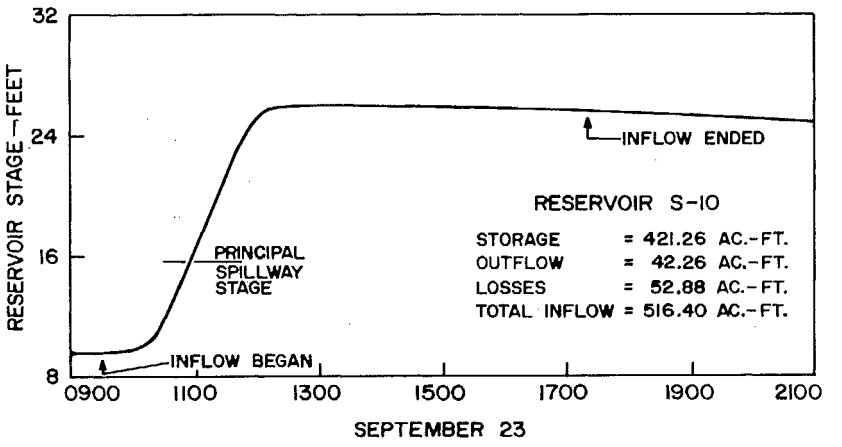


Figure 4. *Time-stage record for reservoir S-10, 23 September 1965*

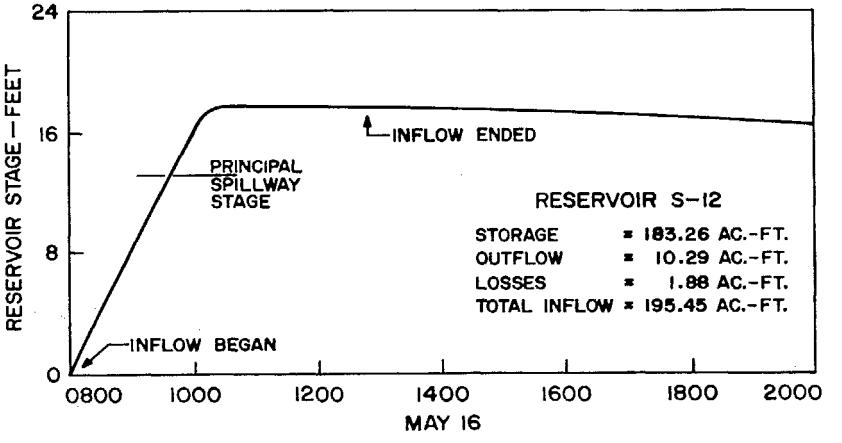


Figure 5. *Time-stage record for reservoir S-12, 16 May 1965*

Examples of hydrographs for condition 2 for reservoirs S-10 and S-12 are shown in Figures 4 and 5, respectively. The estimates of outflow, loss, and total inflow volumes are given in these figures. It is noted that the loss volume for reservoir S-10 is 10.24 per cent of the total inflow. From the above data, it is obvious that a significant error can be made in estimating surface runoff if only the indicated storage is used.

Emergency spillway flow has not been experienced since data collection began. However, the procedure would be similar to that already described and can be expressed as

$$I_t = S_t + \sum_{i=0}^{i=t} L_i + \sum_{i=0}^{i=t} (Qp)_i + \sum_{i=0}^{i=t} (Qe)_i \qquad (3)$$

where Qe is the volume of emergency spillway discharge.

INFLOW AND LOSS COMPUTATIONS BY DIGITAL COMPUTER

The high-speed digital computer can be utilized conveniently to eliminate the numerous calculations necessary to process a single inflow event. The procedure for the electronic computer is slightly different from that outlined previously. The input includes an estimated loss rate and a test procedure is used for each input of stage, time and loss rate to determine the end of inflow. This enables a step-by-step (time-by-time) computation to the end of inflow and does not require an initial determination of the time of the end of inflow. A simplified flow chart for computer processing is shown in Figure 6.

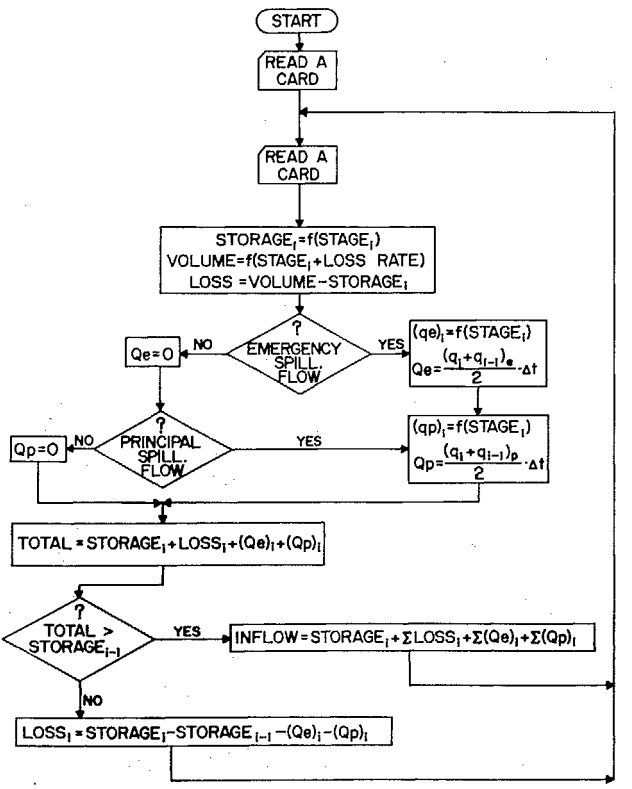


Figure 6. *Generalized flow chart for leaky-reservoir inflow computations*

Benefits of the digital computer, in addition to the elimination of monotonous calculations, include loss computations after the end of inflow. Loss rates other than spillway flow can be calculated from time changes of actual reservoir stage. The availability of a number of records for each reservoir enables statistical analyses of losses. One such analysis has been made. Thirteen independent variables were tested in a factor analysis (Rice, 1967) to determine the best estimators of reservoir loss rate in acre-feet per hour. Reservoir stage alone was found to be the best estimator of volume loss rates and estimating equations were developed. The estimating equations for volume of losses can be used in future computations of inflow instead of the use of loss rate in feet per hour taken from the reservoir stage decline record.

A comparison of the two methods of basic computation of inflow was made for the storm of 9 April 1968, for reservoir S-12 (Fig. 3). Volume losses computed from constant stage loss rates in feet per hour were 2.10 acre-ft. Losses computed by the estimating equation which relates volume loss rate to reservoir stage were 1.99 acre-ft. The difference between the two is approximately 5 per cent. Total inflow by the two methods was 53.97 and 53.86 acre-ft. respectively, or a difference of approximately 0.2 per cent. This is a minor difference and does not greatly affect the accuracy.

SUMMARY

Five floodwater-detention reservoirs in the karstic Edwards Plateau of Texas have been instrumented with bubbler-type gauges to measure inflow. Inflow determination is complicated when high loss rates occur that make it difficult to determine when inflow ends. Inflow determination is further complicated by spillway flow.

Reservoir stage loss rates can be determined from reservoir stage records when inflow is known to have ended. The loss rates can be applied throughout the stage record to estimate total inflow accurately. Laborious calculations can be eliminated by use of the electronic computer. Loss rates estimated from the reservoir depletion record and used as an input into the computer with the time-stage record enables step-by-step computation to the time that inflow ends.

Volume of losses determined from estimating equations agreed within 5 per cent of the volume of losses computed by the conventional method of estimating loss rate in feet per hour from the stage record after inflow ended.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Barron, E. G. 1960. New instruments of the Surface Water Branch, U.S. Geological Survey. A description of the bubbler gage. *Western Snow Conference Proc.* vol. 28, p. 32-38.

Knisel, W. G. Jr. 1965. Groundwater studies in the Edwards Plateau of Texas. *USDA, ARS* 41-100.

Rice, R. M. 1967. Multivariate methods useful in hydrology. *Proc. International Hydrology Symposium,* September 1967, Fort Collins, Colo. U.S.A.

Straub, L. G. *et al.* 1960. Resistance to flow in two types of concrete pipe. St. Anthony Falls Hydraulic Laboratory, *University of Minnesota Technical Paper,* no. 22, series B.

A recording meter for measuring the overland flow

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Abstract. A measuring system is described for determining the overland flow as a component of the stream flow in river beds. The apparatus enables continuous recordings of the overland flow intensity to be made within the range from 2 to 200 l/h. The water flowing off the run-off plot is collected in a trough and led to the recording meter through a device for the removal of eroded soil particles. In the meter two measuring tanks are alternately filled by a tipping tray and emptied by a siphon. The water level changes in the tanks are recorded through floats by two pens on a drum.

APPAREIL ENREGISTREUR DE MESURE DE L'ECOULEMENT DE L'EAU A LA SURFACE DU SOL

Résumé. Cette étude décrit un procédé de mesure servant à déterminer l'écoulement d'eau à la surface du sol comme composante de l'écoulement direct dans un réseau fluvial. L'appareil permet de mesurer et d'enregistrer de façon continue l'intensité d'écoulement dans une gamme de mesure allant de 2 l/h à 200 l/h. L'eau s'écoulant de la parcelle de mesure s'amasse dans un canal d'écoulement et est conduite par un dispositif servant à éliminer les particules provenant de l'érosion du sol. Dans cet appareil, deux coupelles de mesure sont remplies tour à tour à l'aide d'un dispositif à bascule et vidées par des siphons. Par l'intermédiaire de flotteurs, deux plumes enregistrent sur un tambour les changements de niveau d'eau dans les coupelles.

MEDIDOR REGISTRADOR PARA LA MEDIDA DEL FLUJO SOBRE EL TERRENO

Resumen. Se describe un sistema de medición que se utiliza para determinar el flujo sobre el terreno como componente del flujo de corriente en los cauces fluviales. El aparato permite mediciones continuas y con registro de la intensidad de flujo sobre el terreno a realizar dentro de la gama de 2 a 200 litros por hora. El flujo de agua procedente de la parcela de escorrentía se recoge en un efluente y se lleva a un medidor registrador a través de un dispositivo destinado a eliminar las partículas del suelo arrancadas por erosión. En el medidor se llenan alternativamente dos tanques de medición con una bandeja desplazable y se vacían por una tubería con sifón. El nivel de agua cambia en los tanques y se registra, por medio de flotadores mediante dos plumas registradoras, sobre un tambor.

САМОПИСЕЦ ДЛЯ ИЗМЕРЕНИЯ ПОВЕРХНОСТНОГО СТОКА

Резюме. Описывается измерительная система, использовавшаяся для определения поверхностного стока, как одной из составляющих речного стока. Прибор обеспечивал выполнение непрерывных измерений (с записью) интенсивности поверхностного стока в диапазоне от 2 до 200 л/час. Вода, стекающая со стоковой площадки, собирается в лоток и подводится к записывающему прибору, пройдя предварительно через устройство для удаления смытых частиц грунта. В измерителе два бака наполняются поочередно опрокидывающимся лотком опорожняются сифонной трубкой. Изменения уровней воды в баках записываются с помощью поплавков двумя перьями на барабан.

In natural drainage areas transformation from rainfall to run-off takes place in two stages, run-off formation on the surface and run-off concentration in the river bed. Measurements for investigation of the second stage-flow gauging in natural channels have long been made by the utilization of widely differing physical principles. The measurements, however, do not suffice as a basis for hydrological modelling and it is thus necessary to cover quantitatively those processes which take place directly on the surface. We mean such processes as overland flow, representing the main component of the streamflow in the river bed, infiltration, and interflow along defined soil strata. During such investigations the water entering the soil from rainfall and surface run-off will frequently be measured. The rate of infiltration can be determined as the difference between these quantities. These measurements are also a necessary precondition for successful soil erosion research.

The following measuring instrument is especially suitable for investigations of the type mentioned above. Moreover, the method may be employed successfully elsewhere, where the run-off of open channels is determined, for instance, by means of simple gaugings by a tank. It is also used in the determination of the intensity of overland flow in inflitrometer tests.

The principle of the arrangement will now be explained with reference to the schematic drawing in Figure 1; Figure 2 shows a total view of the instrument. The water flowing from the run-off plot is collected in a trough and fills cylinder (2) through the supply line (1). Transfer of the cylinder water level data onto the recording drum (3) is effected via the float (4), rod (5) and recording pen (6). The effective filling capacity of the cylinder is 1 litre. After the cylinder has been filled it is siphoned off (7). In order to obtain a definite quantitative measurement, a second cylinder (2) is filled and measured in the same manner while the first is drained. Switching from one cylinder to the other is done by a tipping tray (8), and tipping is initiated via the rod (5) and lever (9) as soon as the maximum filling height has been attained. Alternate filling of the measuring cylinders is accomplished by having two separate chambers in the receiving tank (10).

The infiltration tests for which the instrument was developed serve the purpose of representing natural storms in models. This criterion defined the dimensions of the instrument, which are such that the maximum values of storm intensity applicable to the climatic range of the Osterzgebirge (Eastern Ore Range near Dresden) could be covered. In terms of the plot of 1 m^2, the limiting values were determined as 200 1/hr for short storm periods and as 2 1/hr for extended storm periods. It was intended to obtain, within these ranges, an approximately constant intensity so that several rotation speeds became necessary. The drum is driven by a 7-day precision clock running via a back gear which permits setting of the following rotation speeds: 1 rotation in 1 hr, 1 rotation in 6 hr and 1 rotation in 24 h. The drum diameter was specified so that the units of time can be measured as integer multiples of 1 millimetre and in this way evaluation of measurements is considerably facilitated if normal graph paper with millimetre squares is used. The chart is fastened to the drum by means of a clip. An auxiliary device is used to start the clock at the beginning of a test.

The draining time of the cylinders (2) must be shorter than their filling time; this necessitates correspondingly large cross-sections in the siphon piping. With small intensities, however, such large siphon piping results in the slowly entering water running off over the apex of the siphon at the same slow speed, without entraining the water present in the measuring cylinder. In any such case, the minimum increase of the water level required for initiating siphoning will be effected by the water

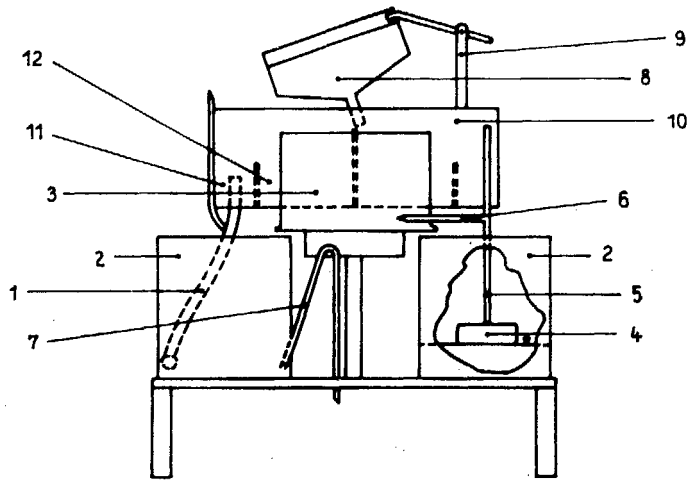


Figure 1. *Principle of the recording meter*

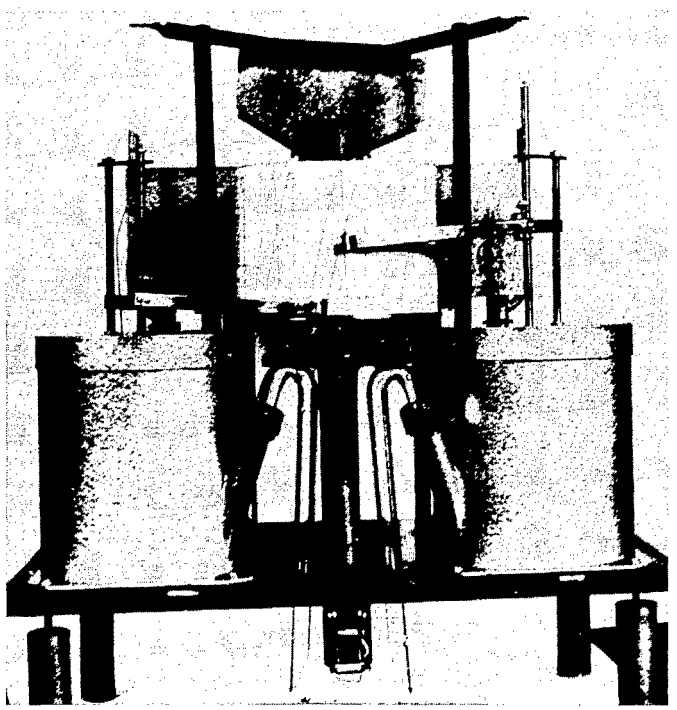


Figure 2. *Total view of the recording meter*

running from the reserve chamber (11) after the tipping tray has been switched over.

Any entrained plant and soil particles may choke the piping system and thus adversely affect the function of the measuring instrument. Coarse particles are retained in a filter in front of the tipping tray while elutriatable soil particles are trapped in a sedimentation basin (12).

The finite storage effect of the instrument is attained especially by the chambers (11) and (12) and dead space in the cylinders (2) so that there is a delay between outflow and inflow. Elimination of this delay requires the start of measurement only after the storage spaces have been filled.

All components are made of non-corrosive material with polished surfaces. The enclosure is fastened to the base plate by two quick-release locking levers. The cover plate is made of Plexiglas and an inspection glass of the same material is incorporated into the front plate.

The instrument detailed above has proved its worth in run-off recording in a series of well-defined laboratory tests as well as in actual measurements *in situ.*

Hydraulic model study to determine a stage-discharge relationship

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Abstract. A large part of the runoff in the Burdekin River at Burdekin Falls in Queensland, Australia, is derived from flood flows which can exceed two million cusecs (57,000 m^3/sec). Because of the difficulty of gauging flows of this order by conventional means, it was decided to derive the stage-discharge relationship by a hydraulic model study.

A model was construct based on a photogrammetreic survey of approximately 3 miles (4.8 km) of the river to an undistorted scale of 1:100. Data necessary for model verification were provided from current meter gaugings for relatively low flows of up to 16,000 cusecs (460 m^3/sec) and flood slopes at discharges of up to about 700,000 cusecs (20,000 m^3/sec) from observations at two stage recorder sites.

The study indicated that the model could produce a stage-discharge relationship to an accuracy of about ± 5 per cent for flows up to 1,000,000 cusecs (28,500 m^3/sec).

MODELE HYDRAULIQUE D'ETUDE POUR LA DETERMINATION DE LA RELATION HAUTEUR / DEBIT

Résumé. Le ruissellement du bassin de la rivière Burdekin aux chutes Burdekin dans le Queensland, en Australie, donne souvent des crues qui peuvent dépasser deux millions de cusecs (57 000 m^3/s.). Devant la difficulté de mesurer ces grands débits par des méthodes conventionnelles, il a été décidé de déduire la relation hauteur / débit d'une étude sur modèle hydraulique.

On a construit un modèle basé sur une étude photogrammétrique d'une longueur d'environ trois milles de la rivière à une échelle sans distorsion de 1/100. Les données nécessaires pour la vérification du modèle ont été fournies par des mesures au mouvement pour des débits relativement faibles allant jusqu'à 16 000 cusecs (460 m^3/s.) et par la mesure de pentes de crues pour des débits allant jusqu'à 700000 cusecs (20 000 m^3/s) à partir d'observations sur deux limnigraphes.

L'étude montre que le modèle peut fournir la relation hauteur/débit avec une précision de ± 5 % pour des débits atteignant un million de cusecs (28 500 m^3/s).

ESTUDIO DE MODELOS HYDRÁULICOS PARA DETERMINAR UNA RELACIÓN NIVEL–CAUDAL

Resumen. Una gran parte de la escorrentía en Burdekin River y en Burdekin Falls, en Queensland, Australia, es producida por caudales de crecidas que pueden exceder los dos millones de pies^3/seg (57 000 m^3/seg). Debido a la dificultad de calibración de caudales por encima de los citados, por medios convencionales, se decidió deducir la relación nivel-caudal por medio de un estudio de modelos hidráulicos.

Se construyó un modelo basado en un estudio fotogramétrico de aproximadamente tres millas de río a una escala no distorsionada de 1:100. Se obtuvieron los datos necesarios para la verificación del modelo a partir de calibraciones con medidores de corriente hasta caudales relativamente bajos del orden de 16 000 pies^3/seg (460 m^3/seg) y pendientes y caudales de crecida hasta unos 700 000 pies^3/seg (20 000 m^3/seg) a partir de observaciones en dos emplazamientos con registradores de nivel.

El estudio indicó que el modelo podría permitir la obtención de una relación nivel-caudal con una precisión de aproximadamente ± 5 % para caudales hasta 1 000 000 pies^3/seg (28 500 m^3/seg.).

ГИДРАВЛИЧЕСКИЕ МОДЕЛЬНЫЕ ИССЛЕДОВАНИЯ ДЛЯ ОПРЕДЕЛЕНИЯ КРИВОЙ РАСХОДОВ

Резюме. Большая часть стока р. Бардекин у порогов Бардекин в Квинсленде обусловлена паводками, которые имеют расход до 2.10^6 ф³/сек. (57 тыс. м³/сек.). Вследствие трудности измерения высоких расходов такой величины обычными средствами, было решено получить кривую расходов путем гидравлических модельных исследований.

Модель была построена на основании фотограмметрической съемки участка реки длиной 3 мили в неискаженном масштабе 1 : 100. Данные, необходимые для продерки модели, были получены на основе вертушечных измерений для сравнительно низких расходов до 16 000 ф³/сек. (460 м³/сек.) и уклонов в период паводка при расходах около 700 тыс. ф³/сек. (20 тыс. м³/сек.) по данным наблюдений на двух самописцах уровня.

Исследование показало, что модель может воспроизвести кривую расходов с точностью ± 5 % для расходов до 1 000 000 ф³/сек. (28,5 тыс. м³/сек.).

INTRODUCTION

The establishment of a stage-discharge relationship by conventional current meter methods for a large seasonal stream which experiences extreme floods can involve considerable difficulty, expense and personal risk. Such a situation exists on the Burdekin river at Burdekin Falls, where streamflow records are required in connexion with an investigation of a large irrigation scheme involving a major dam at the Falls. The 45,000 square mile (117,000 km^2) catchment at this site is subject to tropical cyclones and rain depressions which can result in flood peaks estimated in excess of two million cusecs (57,000 m^2/sec) and large flood volumes which contribute the bulk of the run-off from this catchment.

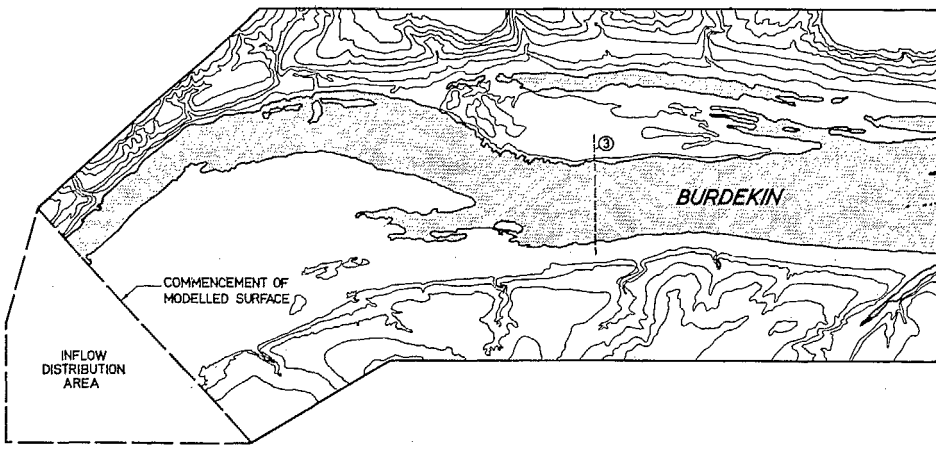
A stream gauging station equipped with an automatic stage recorder was installed just upstream of the Falls where it was considered that a satisfactory stage-discharge relationship could be established by a hydraulic model study. Although little work has been done on establishing such a relationship using models, it was considered that the chances of success for the Falls site were good because the rock bar at the head of the Falls forms a stable control and the Falls are not drowned-out with major floods. A stable relationship was, therefore, expected, with no loop effects in the rating due to channel storage.

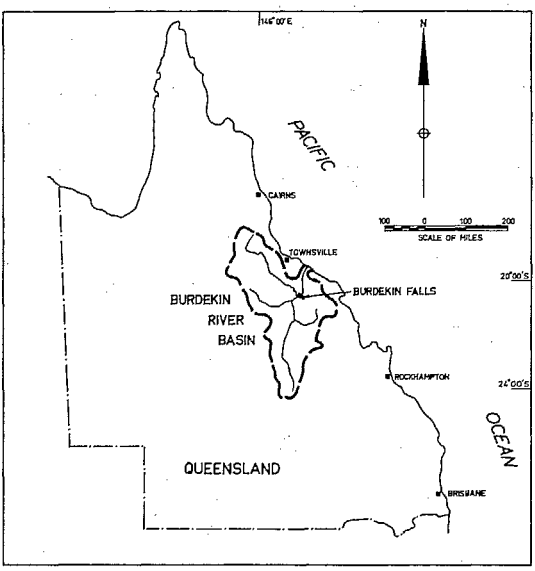
The procedure envisaged was the establishment of the relationship by current meter gaugings up to the level of effective inundation of the Falls, the reproduction of this relationship in the model and the extension of it by reproduction of recorded slopes and surface velocities obtained at higher flows in the reach immediately upstream.

The Snowy Mountains Hydro-electric Authority undertook the laboratory aspects of the model rating of this station for flows between 20,000 (570) and 1,000,000 cusecs (28,500 m^3/sec) for the Irrigation and Water Supply Commission, Queensland; the Commission undertook the collection of hydrographic data necessary for the calibration of the model.

DESCRIPTION OF THE SITE

The Burdekin river, at 99 river miles (159 km) from its mouth, passes over a section known as the Falls, where it drops about 80 ft (26 m) in height in half a mile and


③
BURDEKIN
COMMENCEMENT OF
MODELLED SURFACE
INFLOW
DISTRIBUTION
AREA


146°00'E
N
PACIFIC
CAIRNS
100 0 100 200
SCALE OF MILES
TOWNSVILLE
20°00'S
BURDEKIN FALLS
BURDEKIN
RIVER
BASIN
ROCKHAMPTON
24°00'S
OCEAN
QUEENSLAND
BRISBANE

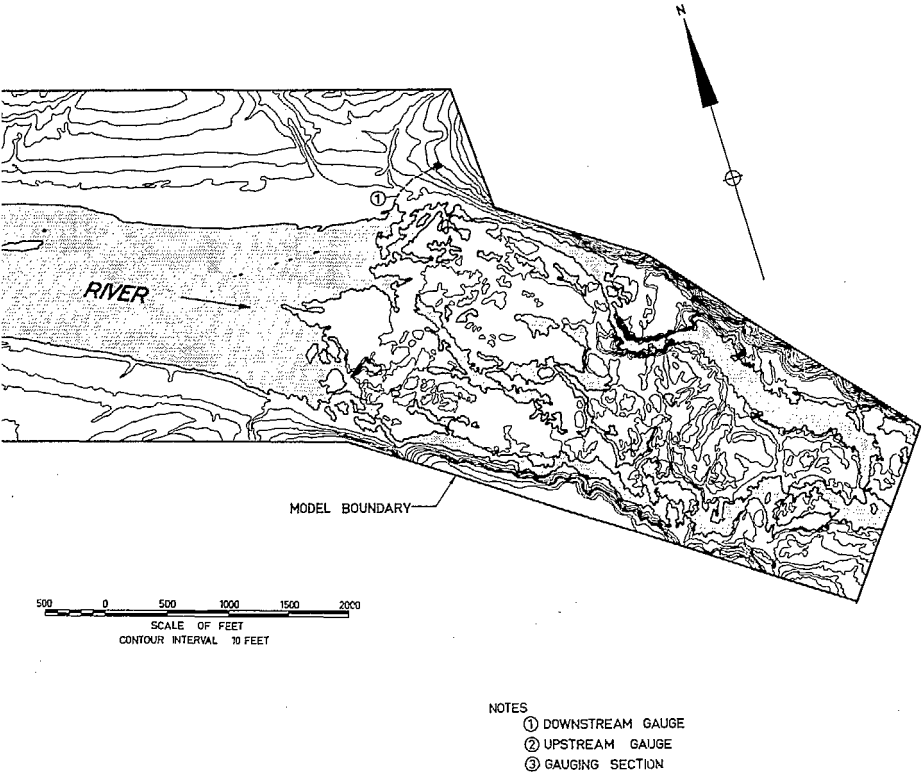


Figure 1. *Plan and location of the modelled area*

then flows on through the Burdekin Gorge for several miles (Fig. 1). The significance of the Falls from the stream gauging aspect is that they remove any variable downstream effects on the stage-discharge relationship at the head of the Falls.

Immediately upstream of the Falls for almost 1 mile (1.6 km) there is a broad shallow pool which is generally clear of obstructions apart from a few timbered ridges and rock bars which outcrop at about 15° to the axis of flow; the height of these outcrops is generally less than 10 ft (3.2 m). Apart from this, the bed material largely consists of sand and gravel, and substantial deposits of this material further upstream indicate that during flooding there is a large bedload movement. It is believed, however, that the bed for at least a mile (1-6 km) upstream of the Falls is generally static, undergoing little change from flood to flood due to the presence of the rock bars. Upstream of this mile-long pool, there is a low, timbered spit along the left bank which forms an island at minor flood flows and is inundated at higher flows.

Apart from the few ridges which outcrop at low flows, the bed is free of trees; the tree cover on the banks at low levels is moderate to dense, while scattered tall trees grow on the higher levels. Details of the configuration of the site are shown of Figure 1.

HYDROGRAPHIC DATA

The basic stage recorder for collection of long-term records was established on the left bank at the downstream end of the large pool at the head of the Falls. This installation is of the gas-purge manometer-servo type and referred to hereafter as the downstream gauge.

The information required for model calibration included current meter gaugings to establish a stage-discharge relationship for discharges up to the stage at which the upstream portion of the Falls was effectively inundated; with flows of this order it was considered that verification of levels and roughness of the Falls could be achieved. It was estimated that a reliable relationship up to 40,000 cusecs (1,100 m^3/sec) would be required and that this might be the practical limit of current meter gauging from a boat. Because the relationship has to be extended from about 40,000 (1,100) to 1,000,000 cusecs (28,500 m^3/sec), it was also necessary to check bank roughness in the model using flow parameters which could be measured in the prototype at much higher flows without the need of using a boat; the parameters selected were float velocities and flood slopes. Float velocity observations were required in the reach immediately upstream of the Falls, together with a slope-stage relationship for discharges up to about half a million cusecs (14,300 m^3/sec). A second stage recorder similar to the first was, therefore, installed on the left bank, 3,500 ft (1,070 m) upstream of the first recorder; this unit is referred hereafter as the upstream gauge.

During a large flood in January 1968, a number of gaugings were obtained from a boat which provided a satisfactory rating up to about 16,000 cusecs (460 m^3/sec). An incomplete gauging attempted at the same time was later estimated to have a discharge of 45,000 cusecs (1,290 m^3/sec). This higher discharge was subsequently used in the model calibration but since it was partly estimated, further gaugings of this order of flow are required. On the falling stage of the flood a number of slope-stage readings were obtained for discharges later estimated to be between 130,000 (3,700 m^3/sec) and the peak of 200,000 cusecs (5,700 m^3/sec).

Debris left in February 1968 by a flood estimated at the time to have a peak discharge of half a million cusecs (14,250 m^3/sec), provided the highest slope-stage measurement, but was of uncertain accuracy, and readings on the gauges during the recession of this flood provided additional data for slope-stage determinations. No float velocity measurements were obtained during these two floods.

MODEL DESIGN AND CONSTRUCTION

Several items were considered in the choice of the scale of the model and of the area to be covered by the model. It was necessary to have a scale which would allow accurate study for low flood discharges of 10,000 (290) to 40,000 cusecs (1,100 m^3/sec) and in addition, allow measurements for a prototype discharge of up to 1,000,000 cusecs (28,500 m^3/sec). For lower discharges, between 10,000 (290) to 15,000 cusecs (430 m^3/sec), it was necessary to ensure that the scale of the model was sufficiently large to provide fully turbulent flow conditions and to produce water depths over the Falls which were not grossly affected by surface tension. The area modelled was required to extend sufficiently past the bend upstream of the Falls to ensure correct approach flow conditions and downstream of the Falls to ensure correct tailwater effects.

As a result of these considerations the model coverage was a shown in Figure 1. Important features shown on this plan include the position of the downstream and upstream gauges and the gauging section used during the January 1968 floods. Froude scaling, without distortion, was used for the model study and the pertinent scales were:

length 1:100
discharge 1:100,000
velocity 1:10
time 1:10 .

Approximately 100 templets cut as cross-sections of the river were placed to allow detailed formation of the surface of the model in cement mortar. Contour plans of the surface above water level, with intervals down to 2.5 ft (0.75 m), were provided from photogrammetric surveys taken when the river was at its normal low level. In addition, isolated levels at cease-to-flow points of a number of channels at the top of the Falls were taken and cross-sections of the river at 500 ft (150 m) intervals for 1 mile (1.6 km) upstream of the Falls were sounded and used for modelling the bed of the river. Details of the river bed beyond 1 mile (1.6 km) upstream of the Falls were not available and bed levels in this region, although not critical, were estimated from known details of rock bars and isolated levels that were available.

The representation of detail in the Falls area was the subject of careful examination. Areas between templets were formed in the normal manner with the aid of the contour plans, but the exact reproduction of the jagged features of the Falls was both impossible und unnecessary; it was important, however, to reproduce the general relief and the effective roughness of the Falls.

Artificial roughness of the surface of the model was necessary for the reproduction of prototype water levels. In the Falls area a gravel size varying from 3/32 (2.4) to 3/16 in (4.8 mm) was used and applied with a cement slurry. Upstream of the Falls, 1/2 in (12.7 mm) mesh chicken wire and rubberised hair sheeting were used to represent vegetation and other bank roughness effects.

Inflow to the model was metered through a 90° triangular weir constructed to conform with recognized standards. Discharge could be varied by vertical movement

of the weir and this provided a facility for the introduction of a hydrograph to the model.

Point gauges located in positions corresponding to the intake to the prototype recorders were used for determining water levels at the upstream and downstream gauge positions. These point gauges allowed measurements to 0.0005 ft (0.15 mm).

Figure 2 shows a general view of the model and Figure 3 shows the gauge and some typical roughness in the model.

MODEL VERIFICATION AND TESTING PROGRAMME

A number of gaugings were obtained for discharges up to 16,000 cusecs (460 m^3/sec). These low gaugings were of value to ensure that levels and roughness on the Falls were correct for flows up to this amount. It was found necessary to make some minor changes to the cease-to-flow levels, but this was not unexpected as there was difficulty in determining these in the prototype.

Roughness was added at higher levels on the upstream section of the Falls so that the downstream gauge recorded correct water levels up to 16,000 cusecs (460 m^3/sec). This was the highest gauging for which a downstream gauge reading was available. At 16,000 cusecs (460 m^3/sec) a portion of the Falls was inundated and model observations revealed that a reliable rating for the downstream gauge up to at least 60,000 (1,700) to 70,000 cusecs (2,000 m^3/sec) would have given more assurance in the verification.

In order to reproduce the water level difference between the upstream and downstream gauges for discharges greater than 16,000 cusecs (460 m^3/sec), extra roughness was necessary in the area upstream of the downstream gauge. This had to be added in such a way that upstream water levels would not be affected for the lower gaugings. For the estimated 45,000 cusec (1,290 m^3/sec) gauging, water was entering vegetation on the banks and wire roughening was added in the lower bank areas in order to reproduce the correct upstream gauge level for this gauging: a downstream gauge reading was not available for this gauging.

Slopes which had been obtained for discharges greater than those gauged, up to an upstream gauge level of 32 ft (9.8 m), showed that additional model roughness was needed to increase the slope of the water surface. Because the slopes at lower discharges were satisfactory, it was apparent that more bank roughness was required, and this was added according to the estimated density of vegetal growth in the prototype as assessed from aerial photographs.

The stage-discharge relationship was required for discharges considerably in excess of those corresponding to the highest slope-stage observations. No float velocities were available and it was necessary, therefore, to use other means of assessing the reliability of the flow pattern and roughness reproduction and the trend of the curves beyond the data coverage. This was done by reproducing all the data with an application of roughness, removing it and repeating the process with a redistribution of the roughness. By ensuring that both distributions of roughness had a feasible conformity with the prototype situation, it was possible to estimate accuracy from the shift of the stage-discharge curves. Initially a combination of rubberized hair and chicken wire mesh was placed upstream of the Falls to reproduce the stage data. When this had been achieved, levels were observed for the desired range of discharges up to one million cusecs (28,500 m^3/sec).

Following the first reproduction of the data an inspection of the prototype was made in which it was possible to gain a detailed impression of vegetation density and the presence of rock outcropping on the banks. It was necessary to have the roughness densities reproduced accurately in the model so that flow distribution effects at the gauges would be reproduced. If this was not achieved, it would be possible to have local velocities at the gauges far removed from what they should be and incorrect water levels due to unnatural wave or surge conditions. Following the prototype inspection the use of rubberized hair to represent dense bank growth was discontinued as it was considered to be a little too dense. Various thicknesses and arrangements of wire mesh were used to provide the second reproduction of the prototype data. A little of this roughness was considered necessary adjacent to and just downstream of the downstream gauge on the left bank.

RESULTS

Stage-discharge relationship

All of the prototype data were reproduced in the model. The upstream and downstream levels were recorded for a large number of discharges to accurately define the stage-discharge curves. Figure 4 shows the curves for both gauges for discharges up to one million cusecs (18,500 m^3/sec) for both roughness distributions. The prototype gaugings of January 1968 are also shown. It will be noticed that the curves tend to divergence from a discharge of approximately 160,000 cusecs (4,600 m^3/sec). An interesting feature, however, is that the redistribution of the bank roughness, although substantial, produced a shift of only 4 per cent for discharges between 300,000 (8,600 m^3/sec) and 700,000 (20,000 m^3/sec) reducing to about 2,5 per cent at one million cusecs (28,500 m^3/sec).

For both distributions of roughness the model accurately reproduced the slope data. The slope-stage relationships obtained from the model for the two roughness distributions showed only a very slight separation, which indicated that changes in the flow patterns at the gauge did not significantly affect readings. It is believed that the roughness in the final arrangement more closely conformed to the relative density of roughness in the prototype and it was recommended that for prototype use the upper curves be used.

An interesting product of the study was that the flood experienced in February 1968, with a peak of 27 ft (8.2 m) on the downstream gauge, had a corresponding discharge of 690,000 cusecs (19,700 m^3/sec).

Reliability of the model reproductions

In a project such as this there are many sources of error, some of a minor magnitude and others which could be of a major magnitude. Possible sources of error in any prototype gauging situation would be inaccuracies in the setting and reading of the gauges, inaccuracies due to calibration of the current meter, assumptions in the gauging procedure, and difference between computed and actual areas. Possible sources of error in the model are the inflow measurements, incorrect location for the water level measurements, and inaccuracy in the representation of roughness.

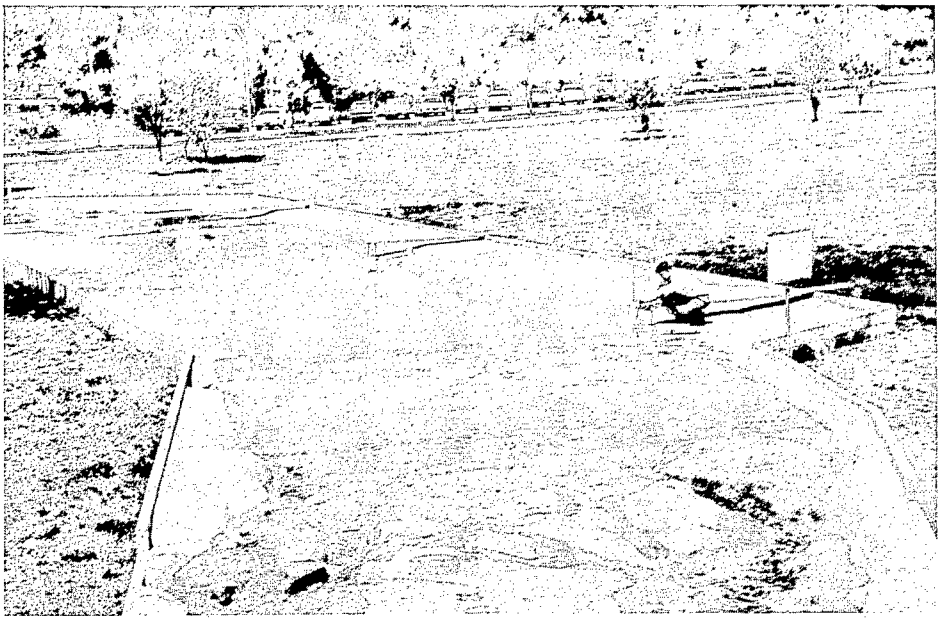


Figure 2. *General view of the model*



Figure 3. *Gauges and typical roughness*

In order to assess the accuracy of the Burdekin Falls model, possible errors in the prototype information and the likelihood of having insufficient prototype data for verification purposes have been disregarded.

The possible error of inflow measurement to the model was believed to be of the order of 1 per cent. Little change in over-all flow patterns occurred from the first to the second distribution of roughness and it is considered that inaccuracies in water level measurement due to local peculiarities in flow conditions were negligible. Two of the three major roughness subdivisions, on the Falls and in the bed of the river, were the subject of alteration and verification using discharge gaugings up to 16,000 cusecs (460 m^3/sec) and stage data for these gaugings for both the upstream and the downstream gauges. The estimated 45,000 cusec (1,290 m^3/sec) gauging was used for verifying the stage height at the upstream gauge, but this could not be done at the downstream gauge because the stage was not obtained for this discharge and could not be derived with sufficient accuracy by correlating available data. The model, therefore, has only been effectively verified for flows up to 16,000 cusecs (460 m^3/sec), which gave partial inundation of the Falls. This introduced a possible source of error involving the balance of roughness between the bed of the river and on the Falls for flows between 16,000 cusecs (460 m^3/sec) and 70,000 cusecs (2,000 m^3/sec).

For the purposes of this study, further prototype gaugings up to 60,000 (1,700) to 70,000 cusecs (2,000 m^3/sec) and float velocity measurements at higher discharges are necessary to confirm the verification of the model.

Whereas the first distribution of roughness was determined to have too great a contrast between dense and sparse vegetation, the second was aimed at reproducing true conformity with the prototype. From this aspect of the reproduction, the 4 per cent maximum shift in discharge which resulted from the redistribution of roughness was considered to be in excess of a reasonable tolerance one could place on the stage-discharge curves.

In this study the presence of the Falls acting predominantly as a form rather than a frictional flow control, substantially increases the confidence which may be placed on the results. In the light of this and the previous discussion, the model reproduction of available data could be expected to have a tolerance of ± 5 per cent.

CONCLUSIONS

The upper curves in Figure 4 may be used as stage-discharge relationships for the prototype of water levels measured at the downstream and upstream gauge sites. These curves may be considered to have a tolerance of ± 5 per cent when assessed on present knowledge of the prototype. The accuracy of the stage-discharge relationship can only be confirmed when the required data for verification of the model have been obtained.

The Burdekin river at Burdekin Falls modelled in this study is fairly unique as far as stream gauging sites are concerned and, as indicated above, the configuration of the site increased the confidence in the results. There is a need, however, to determine the stage-discharge relationships for the more general type of stream in Australia, which has a movable sandy gravel bed and in which the relationship is controlled by channel friction. Consideration is, therefore, being given to carrying out further hydraulic model work on such a stream, the Gascoyne river in Western Australia, under the auspices of the Australian Water Resources Council. In this work

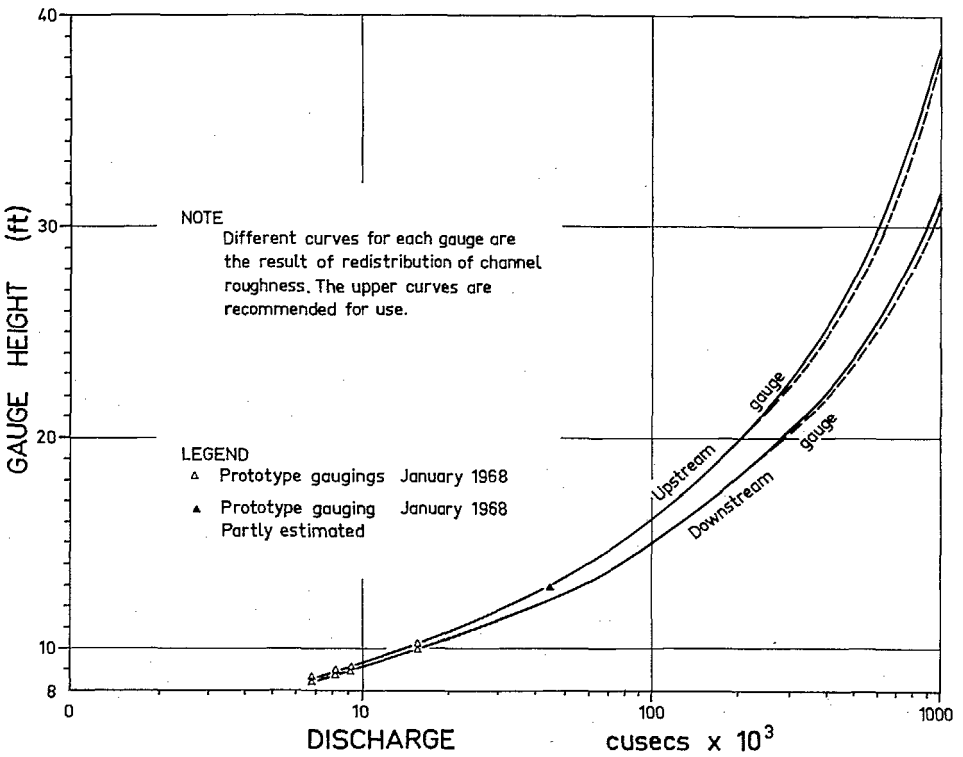


Figure 4. *Stage-discharge relationships*

it is proposed to collect prototype data on discharge using a cableway gauging facility and current meters for flows up to an expected maximum of about 300,000 cusecs (8,600 m^3/sec), surface velocities, flood slopes, bed configuration during floods, bed material, suspended sediment data and water temperatures. The requirements for this proposed work are more demanding than those for the Burdekin Falls study and the work is made more difficult by variable bed friction during floods. The aim is to determine the accuracy of such model work and the prototype data required to give an acceptable accuracy of the stage-discharge relationship for computing stream flows, so that further model work on other similar streams could follow.

ACKOWLEDGEMENTS

The authors are indebted to their respective organizations for permission to prepare this paper and to the officers involved with the data collection and the model work. In particular, they wish to acknowledge the detailed model work which was carried out by Mr. E. J. Lesleighter and the assistance which he and Mr. K. A. Smith, officers of the Snowy Mountains Hydroelectric Authority, gave in preparing this paper.

Problems of flow measurement in large reservoirs

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Abstract. The backwater effect of a high dam or of a series of dams can influence the flow in a very long reach of river. Direct measurement of the velocity or discharge in such streams is rather complicated; it can give good results only under simple conditions of flow through the reservoir.

In order to illustrate this complicated situation, a simplified analysis of the longitudinal distribution of discharge in a reservoir is presented. If a tributary joins the main stream within the backwater reach, the rate of flow and the direction of flow in any cross-section depend on the inflow from the main stream and from the tributary, on the outflow from the reservoir and on the geometric and hydraulic characteristics of the reservoir.

The analysis assumes that the temperature and density of water are homogeneous and that the changes in the inflow and outflow rates are slow enough to avoid formation of waves and secondary currents in the reservoir.

PROBLÈMES DE MESURE DE L'ÉCOULEMENT DANS LES GRANDS RÉSERVOIRS

Résumé. Le remous dans un réservoir ou dans une série de réservoirs sur une rivière peut influencer le mouvement d'eau sur un long bief. La mesure du débit dans de tels cours d'eau est très compliquée ; elle ne peut donner de résultats satisfaisants que dans des conditions d'écoulement simples dans le réservoir.

Pour éclairer un peu la situation, cette étude présente une analyse simplifiée de la distribution longitudinale du débit dans un réservoir. Si un cours d'eau tributaire tombe dans la zone de remous de la rivière principale, le débit et la direction du mouvement d'eau dans une section transversale dépendent du débit dans le cours d'eau principal, du débit dans le cours d'eau tributaire, de l'écoulement du réservoir et des caractéristiques géométriques et hydrauliques du réservoir.

On suppose dans cette analyse que la répartition de la température et de la densité de l'eau sont homogènes et que les variations de débit sont suffisamment lentes pour ne pas causer des ondes et des courants secondaires.

PROBLEMAS DE MEDICIÓN DE CAUDAL EN GRANDES EMBALSES

Resumen. El efecto de las curvas de remanso en una presa influenciada por una serie de presas puede influir en el caudal en un largo tramo del río. La medición directa de la velocidad o del caudal en dichos cursos de agua es bastante complicada, pudiendo dar buenos resultados sólo con condiciones simples de flujo a través del embalse.

Para ilustrar este problema, se presenta un análisis simplificado de la distribución longitudinal del caudal en un embalse. Si un afluente desemboca en el curso principal dentro del campo de la curva de remanso, el valor del caudal y la dirección del mismo en cualquier sección transversal depende del caudal del curso principal y del afluente, sobre el flujo de salida del embalse y sobre las características geométricas e hidráulicas de este último.

Se ha supuesto que la temperatura y la densidad del agua son homogéneas y que los cambios en los flujos de llegada y de salida son lo suficientemente lentos para evitar la formación de olas y corrientes secundarias en el embalse.

ВОПРОСЫ ИЗМЕРЕНИЯ РАСХОДОВ В БОЛЬШИХ ВОДОХРАНИЛИЩАХ

Резюме. Подпорный эффект от высокой плотины или серии плотин может оказывать влияние на поток на весьма длинном участке реки. Непосредственное измерение скорости или расхода в таких реках довольно сложно, оно может дать хорошее результаты только при простых условиях движения потока через водохранилище.

С целью иллюстрации усложненной ситуации будет предложен упрощенный анализ продольного распределения расхода в водохранилище. Если некоторый приток впадает в главную реку в пределах подпорного участка, величина раскода и направление потока в некотором поперечном сечении зависят от поступления воды от главном реки и притока, от сброса из водохранилища и от геометрических и гидравтических характеристик водохранилища.

Анализ предполагает, что температура и плотност воды однородны и что интенсивности притока и оттока достаточно малы, чтобы могли образовываться волны и вторичные течения в водохранилище.

Assume that a symmetrical reservoir with a constant bottom slope has a general shape of cross-section expressed by the equation

$$h = A\left(\frac{b}{2}\right)^n \tag{1}$$

where h is the depth of water; b is the width of the water surface; n is an exponent expressing the shape; A is a constant characterizing the relative depth.

Assuming a hydrostatic (horizontal) water level, the depth along the longitudinal axis of the reservoir can be expressed as

$$h_x = J(L - x) \tag{2}$$

where J is the slope of the bottom; x is the distance from the initial cross-section from the dam (where $x = 0$); L is the length of the reservoir from $x = 0$ up to $x = L$ where $h_x = 0$; h_x is the depth of the reservoir at the point x on the longitudinal axis.

In the following description, the indices 0, L, and x will indicate the values of the initial cross-section, the final cross-section, and a general cross-section at a distance x, respectively. There are then three possibilities:

$Q_L = Q_0$ characterizes a steady non-uniform flow

$Q_L > Q_0$ characterizes the filling of the reservoir; the discharge decreases along the axis of the reservoir from Q_L to Q_0

$Q_L < Q_0$ characterizes the emptying of the reservoir; the discharge along the axis increase from Q_L to Q_0.

Assuming a hydrodynamic water level, the depth in the longitudinal axis of the reservoir can be expressed by

$$H_x = h_x + \Delta h_x \tag{3}$$

where Δh_x represents the backwater effect which can be expressed by a general function

$$\Delta h_x = f(x, Q_L, Q_0, h_0). \tag{4}$$

The total depth is given by a similar function

$$H_x = f(x, Q_L, Q_0, h_0). \tag{5}$$

An infinitesimal increase or decrease of the depth dh_0 (caused by filling or emptying) results in a similar change H_x which can be written as

$$\frac{dH_x}{dh_0} = f(x, Q_L, Q_0, h_0) . \tag{6}$$

Inserting the finite values a_0 and a_x instead of the infinitesimal expressions dh_0 and dH_x, we may write

$$a_x = a_0 . f(x, Q_L, Q_0, h_0). \tag{7}$$

Graphical illustration of the functions 2, 3, 5 and 7 is given in Figure 1. In deep reservoirs, where h_0 is several times greater than the original depth of the river h, the value of a_x will be equal or near to a_0 in most of the reservoir. The decrease of a_x will be evident only in the upper part of the reservoir and it will equal zero theoretically at infinity. The variation of a_x becomes more evident only in smaller reservoirs with a decreasing h_0/h ratio; we are not concerned with this case here.

The total change of volume within the reach between $x = 0$ and $x = x$ corresponding to the change a_0 is given by

$$\Delta V_{0x} = \int_0^x a_x . b_x . dx \tag{8}$$

where b_x can be computed from equation 1:

$$b_x = 2 \left(\frac{h_x}{A}\right)^{1/n} . \tag{9}$$

The term "discharge Q_x" in any cross-section x means that a quantity of water moves during a unit of time from the part of the reservoir before the cross-section x into the part behind this cross-section. In order to simplify the following descriptions, we shall introduce the expression

$$q_x = Q_x - Q_0 . \tag{10}$$

The variation of q_x along the reservoir can be expressed by

$$q_x = q_L \frac{\Delta V_{0x}}{\Delta V_{0L}} \tag{11}$$

hence,after inserting equation 8

$$q_x = q_L \frac{\int_0^x a_x . b_x . dx}{\int_0^L a_x . b_x . dx} . \tag{12}$$

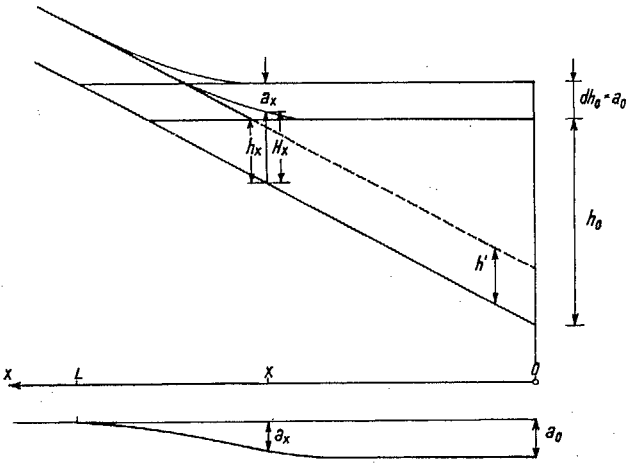


Figure 1. *Graphical illustration of functions h_x, H_x and a_x*

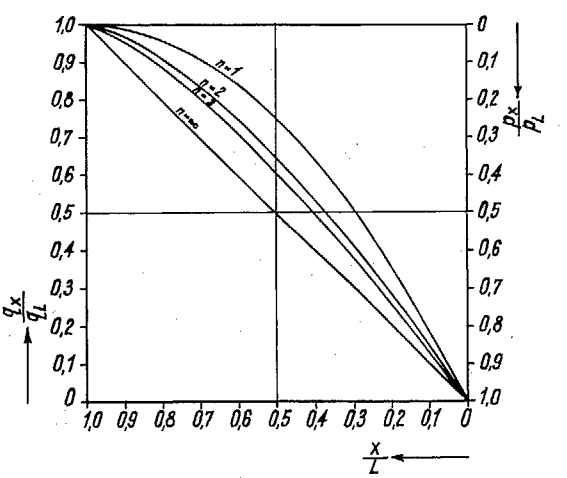


Figure 2. *Emptying and filling a reservoir for four simplified shapes*

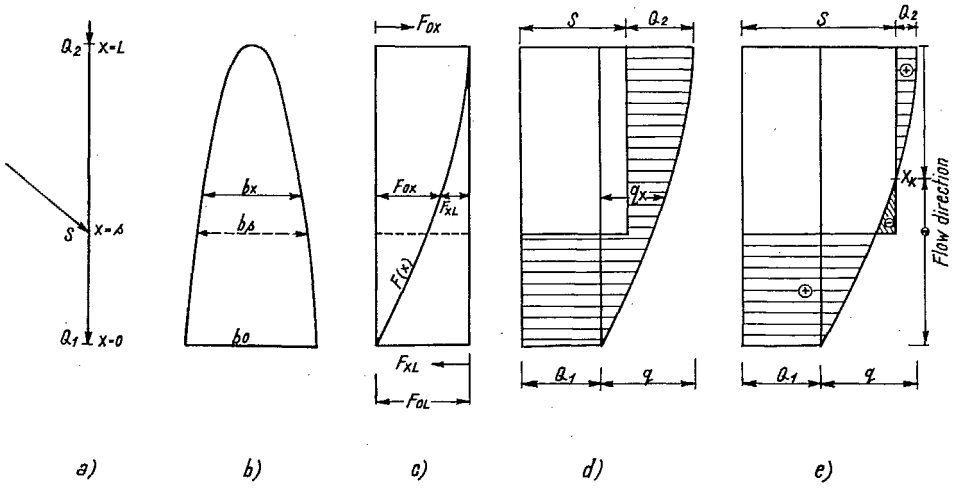


Figure 3. *Different possibilities during filling of a reservoir (for explanation of situations a-e see text)*

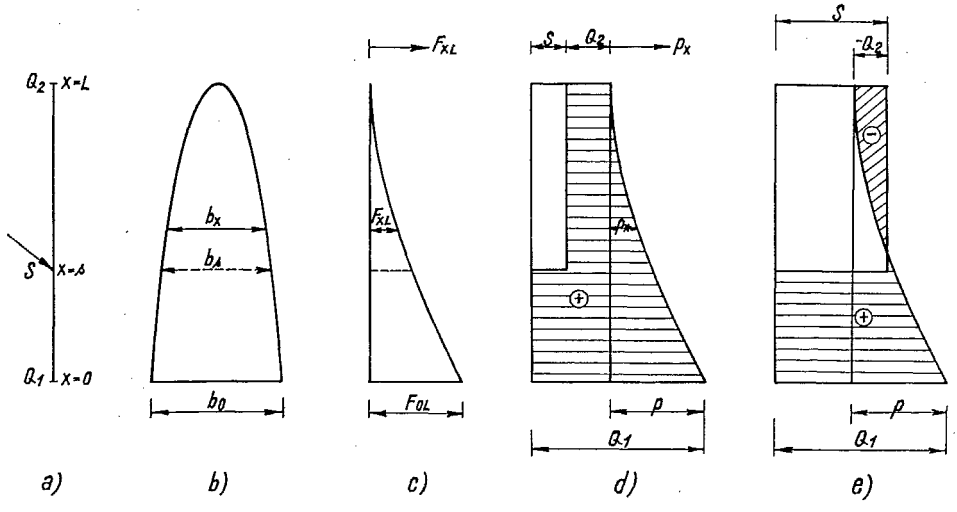


Figure 4. *Different possibilities during emptying of a reservoir (for explanation of situations a-e see text)*

The required discharge Q_x will be obtained from equation 10 by inserting equations 12, 7,and 9. Function 7 is rather complicated and its analytical expression would be very difficult. In order to simplify further analysis, we shall introduce an approximation for deep reservoirs

$$a_x = \text{constant} = a_0. \tag{13}$$

The deviation of expression 13 from the correct value (equation 7) will increase with increasing x. This deviation will only have a practical importance in the upper part of the backwater reach.

We can assume that the backwater effect has no essential influence on the area of the water surface. This assumption can be accepted for most deep reservoirs.

Inserting equations 9 and 13 into equation 12, we obtain, after integration and rearrangement,

$$q_x = q_L \cdot \left[1 - \left(1 - \frac{x}{L} \right)^{\frac{n+1}{n}} \right]. \tag{14}$$

Inserting equation 14 into equation 10, we obtain the required equation

$$Q_x = Q_0 + (Q_L - Q_0) \cdot \left[1 - \left(1 - \frac{x}{L} \right)^{\frac{n+1}{n}} \right] \tag{15}$$

from which the discharge in any cross-section can be computed.

A similar procedure can be used for the emptying of the reservoir when $Q_L < Q_0$. We can introduce the expression:

$$P_x = Q_x - Q_L. \tag{16}$$

The change of depth a_0 causes within the reach between $x = x$ and $x = L$ a change of volume

$$V_{xL} = \int_x^L a_x \cdot b_x \cdot \mathrm{d}x \tag{17}$$

corresponding to the discharge at x. We can write

$$P_x = P_0 \frac{\Delta V_{xL}}{\Delta V_{0L}} \tag{18}$$

or

$$P_x = P_0 \frac{\int_x^L a_x \cdot b_x \cdot \mathrm{d}x}{\int_0^L a_x \cdot b_x \cdot \mathrm{d}x}. \tag{19}$$

Inserting equations 7 and 9, and integrating equation 19, we obtain

$$P_x = P_0 \left(1 - \frac{x}{L} \right)^{\frac{n+1}{n}}. \tag{20}$$

Table 1. *Computation of equations 22 and 23 for four simplified shapes*

$\frac{x}{L}$	Filling				Emptying			
	$n = 1$	2	3	∞	1	2	3	∞
0.0	0.000	0.000	0.000	0.000	1.000	1.000	1.000	1.000
0.1	0.190	0.146	0.131	0.100	0.810	0.854	0.869	0.900
0.2	0.360	0.284	0.257	0.200	0.640	0.716	0.743	0.800
0.3	0.510	0.414	0.378	0.300	0.490	0.586	0.622	0.700
0.4	0.640	0.535	0.494	0.400	0.360	0.465	0.506	0.600
0.5	0.750	0.646	0.603	0.500	0.250	0.354	0.397	0.500
0.6	0.840	0.747	0.705	0.600	0.160	0.253	0.295	0.400
0.7	0.910	0.836	0.799	0.700	0.090	0.164	0.201	0.300
0.8	0.960	0.911	0.883	0.800	0.040	0.089	0.117	0.200
0.9	0.990	0.968	0.954	0.900	0.010	0.032	0.046	0.100

Inserting equation 20 into equation 16, we obtain

$$Q_x = Q_L + (Q_0 - Q_L) \cdot \left(1 - \frac{x}{L}\right)^{\frac{n+1}{n}}. \tag{21}$$

The discharge in any cross-section can be computed from this equation.

The additional discharges q and p, from equations 14 and 20 respectively, can be expressed in dimensionless values. We then obtain for a filling reservoir

$$\frac{q_x}{q_L} = 1 - \left(1 - \frac{x}{L}\right)^{\frac{n+1}{n}} \tag{22}$$

and for an emptying one

$$\frac{P_x}{P_0} = \left(1 - \frac{x}{L}\right)^{\frac{n+1}{n}}. \tag{23}$$

The relative expression 22 and 23 have been computed for the most common simplifications of the shape of the valley, that is for $n = 1$ (triangular cross-section), $n = 2$ (quadratic parabola), $n = 3$ (cubic parabola), $n = \infty$ (rectangular). The results are presented in Table 1 and in Figure 2.

THE DISTRIBUTION OF DISCHARGE IN A RESERVOIR WITH A LATERAL TRIBUTARY

Assume that a reservoir exists with inflow Q_2 at the upstream end and outflow Q_1 at the downstream end. In addition a tributary with inflow S enters the reservoir at the point $x = s$.

The filling of the reservoir is characterized by the expression $Q_2 + S > Q_1$ and emptying by $Q_2 + S < Q_1$. It is evident that the longitudinal variation of discharge in such case will be much more complicated than in the case of one inflow only. According to the relationships between Q_1, Q_2 and S, we can find reaches in the reservoir with both positive and negative directions of flow.

During filling of the reservoir

This condition is characterized by $Q_2 + S > Q_1$ and the different possibilities are illustrated in Figure 3. Figure 3*a* represents the axes of the reservoir and of the tributary. Figure 3*b* represents the water surface, where bx is the width of the water surface at the cross-section x. Figure 3*c* expresses the variation of the area of the water surface as a function of x. The horizontal co-ordinates to the left of the curve $F(x)$ represent the area in the reach from $x = 0$ to $x = x$, the co-ordinates to the right of the curve represent the area between x and $x = L$. We also assume the backwater effect has no influence on the area of a large reservoir.

The increase in volume per unit time in the whole reservoir is given by:

$$q = S + Q_2 - Q_1. \tag{24}$$

The increase of volume along the reservoir (discharge q_x) is assumed to be proportional to the distribution of the surface area. Then, from Figures 3c and 3d, the following equations apply:

$$q_x = q \frac{F_{0x}}{F_{0L}} \tag{25}$$

or

$$q - q_x = q \frac{F_{xL}}{F_{0L}}. \tag{26}$$

The actual discharge within the reach $x = 0$ as far as $x = s$ is given by :

$$Q_x = Q_1 + q_x \tag{27}$$

and within the reach $x = s$ as far as $x = L$ by

$$Q_x = Q_1 + q_x - S. \tag{28}$$

The actual discharge is represented in Figure 3*d* by the dark area. The rate of discharge in this case is positive for the whole reservoir, while flow is from the upstream to the downstream end. In the case of a larger S and a smaller Q_2, a special situation can occur in part of the reservoir with $S > Q_1 + q_x$, while the expression 28 will be negative. This case is illustrated in Figure 3*e*. The flow in the upstream part of the reservoir is positive, the inflow Q_2 supplies the increase of volume up to the point x_k. The discharge in this cross-section is then zero. In the next reach between x_k and $x = s$, the discharge is negative, as it is supplied by the tributary S. In the last part of the reservoir $x = s$ to $x = 0$, the direction of flow is again positive; the increase of volume is fully supplied by the second part of the inflow S.

In the case $Q_2 = 0$, negative flow occurs in the whole upper part of the reservoir upstream of the point $x = s$.

Another extreme case is $S = 0$; but the situation becomes much simpler and the conditions correspond to those described in the first part of this paper.

In the case $S = Q_1 + q_s$, the discharge just above $x = s$ is zero, the discharge just below this point equals S.

The longitudinal distribution of discharge will never be continuous at the point $x = s$ (except in the case $S = 0$), while the change of flow in this cross-section always equal S.

During emptying of the reservoir

This occurs when $Q_1 > Q_2 + S$. The situation is illustrated in Figures 4a, b, c and the following equation applies

$$P = Q_1 - S - Q_2. \tag{29}$$

Then

$$P_x = P\frac{F_{xL}}{F_{0L}}. \tag{30}$$

The actual discharge within the reach between $x = 0$ and $x = s$ is given by the equation

$$Q_x = S + Q_2 + P_x. \tag{31}$$

within the reach from $x = s$ to $x = L$ by

$$Q_x = Q_2 + P_x. \tag{32}$$

A simple case with a positive Q_2 is illustrated in Figure 4*d*. The flow in the whole reservoir is positive.

A special case is presented in Figure 4*e*, The value of Q_2 is negative; there is no inflow at the upstream end of the reservoir, the water is here drawn out of the reservoir. In this case, the flow in the upper part of the reservoir is negative, in the lower part it is positive. The cross-section with $Q = 0$ will occur at the point where $Q_2 = P_x$.

CONCLUSIONS

The analysis presented in this paper illustrates the complications which occur only under simplified and very favourable conditions of measurement. In addition to these complications, we have to add in most reservoirs the effect of other factors, e.g. the density currents caused by the variable density of water due to temperature or sediment, the waves caused by sudden changes of inflow and outflow (e.g. in reservoirs with hydroelectric power production) or by the effect of wind, the irregular shape of the reservoir and the influence of tributaries. These factors complicate not only the measurement of the velocity or discharge, but also the determination of the representative water stage and the instantaneous volume of water stored in the reservoir.

These conditions were met when studying some problems for a series of large reservoirs on the Vltava River. There we found that it is very difficult to obtain a direct measurement of the velocity or the discharge through a cross-section in a reservoir. This measurement cannot be obtained by the usual hydrometric methods for it requires special measuring devices. The purpose of this paper is to draw the attention of hydrologists and manufacturers of the measuring devices to these problems, and to encourage them in studying and developing proper methods and the instruments necessary for such measurements.

LE (Leading Edge) flowmeter – a unique device for open channel discharge measurement

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Abstract. This paper describes the capability of LE acoustic flowmeters to measure open channel volumetric flow. Problems associated with the inhomogeneity of the medium have been overcome, as evidenced by the successful operation of the LE flowmeter on various rivers and canals in the United States.

Using the time differences resulting from transmitting signals between transducer pairs, average path velocities are calculated. Volume flow is computed by appropriately weighting the individual path velocities. Minimum resolvable flow, which is inversely proportional to path length, is about 0.2 m/sec for a 1 m path.

For maximum accuracy, multiple paths are required. In channels with constant stage, flow is obtained by quadrature integration independent of the velocity profile. With variable stage, a water level sensor is also required. A typical flow with stage changes of + 21 per cent, – 7 per cent has been integrated to better than 1 per cent accuracy.

LE COMPTEUR DE DEBIT LE (LEADING EDGE) FLOWMETER, NOUVEL INSTRUMENT POUR LA MESURE DU DÉBIT VOLUMÉTRIQUE DANS UN CANAL DÉCOUVERT

Résumé. Cet exposé décrit les possibilités du LE Leading Edge flowmeter, instrument acoustique de mesure de débit dans un canal découvert. Les problèmes des techniques acoustiques liés à l'hétérogénéité du milieu ont été surmontés, comme le montre le succès de l'opération du LE flowmeter sur divers canaux et rivières des Etats-Unis.

En utilisant la différence de la durée du trajet d'émissions entre une paire de capteurs-émetteurs acoustiques, on calcule la moyenne sur un trajet donné. Le débit est obtenu par un calcul pondéré utilisant les vitesses mesurées dans diverses directions. La résolution minimale de la mesure du débit, qui est inversement proportionnelle à la longueur du trajet, est 0,02 m/s pour un trajet de 1 mètre.

Pour une précision maximale, des mesures dans des directions multiples sont nécessaires. Dans les canaux à niveau fixe, le débit est obtenu par une intégration en quadrature, indépendamment du profil de vitesse. Dans le cas de niveau variable, un senseur de niveau est nécessaire. Un débit typique avec variations de niveau de + 21 % – 7 % a été intégré avec une précision supérieure à 1 %.

EL MEDIDOR DE CAUDAL "LE" (LEADING EDGE), DISPOSITIVO ÚNICO PARA LA MEDICIÓN DE CAUDALES EN CAUCES ABIERTOS

Resumen. El presente trabajo describe la capacidad de los medidores de caudal acústicos LE para realizar mediciones de flujo volumétrico en cauces abiertos. Se han superado los problemas relacionados con la no homogeneidad del medio, por el satisfactorio funcionamiento de los medidores de caudal LE en diversos ríos y canales de los Estados Unidos de América.

Utilizando las diferencias de tiempo resultantes de la transmisión de señales entre pares transductores, se han calculado las velocidades medias de desplazamientos. El flujo volumétrico se calcula por ponderación adecuada de las velocidades de desplazamiento individuales. El flujo mínimo medible, que es inversamente proporcional a la longitud del desplazamiento, viene a ser de 0,02 m/seg., para un desplazamiento de 1 m.

Para obtener la máxima precisión, es necesario utilizar desplazamientos múltiples. En cauces de nivel constante, el caudal se obtiene por integración con independencia del perfil de velocidades. Con un nivel variable, se precisa también un detector de nivel de agua. Se han calculado caudales con cambios de nivel de + 21 % – 7 % con una precisión superior al 1 %.

РАСХОДОМЕР « LE » (РАБОЩАЯ КРОМКА) — УНИКАЛЬНОЕ УСТРОЙСТВО ДЛЯ ИЗМЕРЕНИЯ РАСХОДА В ОТКРЫТОМ РУСЛЕ

Резюме. В докладе дается описание акустических расходомеров « LE » для измерения объема стока в открытом русле. Были преодолены проблемы, связанные с неоднородностью среды, о чем свидетельствует успешное действие расходомера « LE » на различных реках и каналах США.

Рассчитываются средние скорости по траектории с использованием разницы во времени, образующейся в результате передающих сигналов между двумя датчиками. Объем стока рассчитывается путем соответствующего взвешивания отдельных скоростей по траектории. Минимально допустимое течение, которое обратно пропорционально длине траектории, составляет около 0,02 метра/сек на 1 м траектории.

Для получения максимальной точности необходимо располагать многочисленными траекториями. В руслах с постоянным уровнем довы сток рассчитывается по интегрированию квадратуры независимо от профила скорости. Пци переменном уровне воды необходимо также иметь самописец уровня воды. Типичный поток с колебаниями уровня от + 21 % до — 7 % амплитуды интегрируется с точностью до 1 %.

INTRODUCTION

The Westinghouse experience in acoustic velocity meters dates back some ten years, when the first application was for use in the sea environment. Over fifty systems were built and successfully used. It was from that beginning that the LE acoustic flowmeter system originated for discharge measurement. It has since been applied to the measurement of discharge in hydroelectric penstocks up to 8 m in diameter, and in conduits and tunnels from 0.6 to 10 m in diameter. It has demonstrated accuracies of better than 1 per cent when compared with the Gibson-pressure momentum, the Alan salt velocity, and weigh tank methods of measuring discharge. For open stream flows, it has been applied to the measurement of natural rivers as wide as 340 m and as deep as 35 m, and to man-made canals of rectangular and trapezoidal cross-sections. The performance has shown accuracies of 0.5 per cent when compared with conventional current meter stream gauging for the man-made shapes and accuracies of nominally 1 per cent for natural rivers, with occasional variations of up to 4 per cent.

OPERATING PRINCIPLE

Ultrasonic signals travel at a rate that is dependent on the propagation velocity in the medium and on the flow velocity of the medium relative to the direction of signal travel. A typical configuration for a single measurement path system and a simplified block diagram of the LE flowmeter is illustrated in Figure 1. The measurement path is installed at an oblique angle with respect to the fow velocity so that the transit times between the two transducers are given by the expressions

$$T_{AB} = \frac{L}{C + V_F \cos\theta} \quad T_{BA} = \frac{L}{C - V_F \cos\theta} .$$

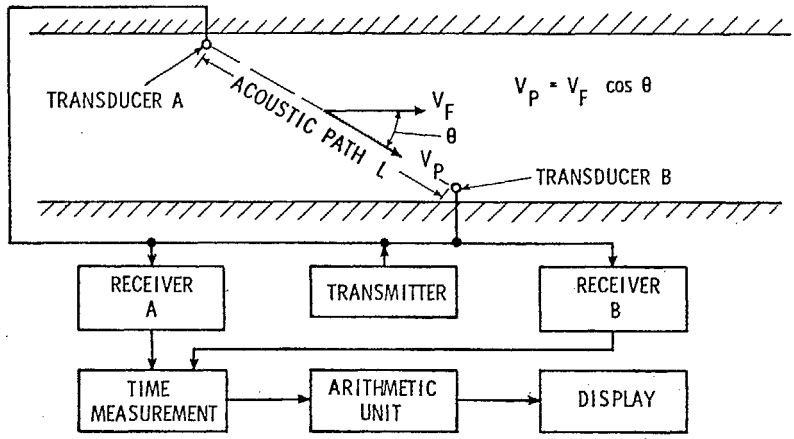


Figure 1. *Typical LE flowmeter configuration*

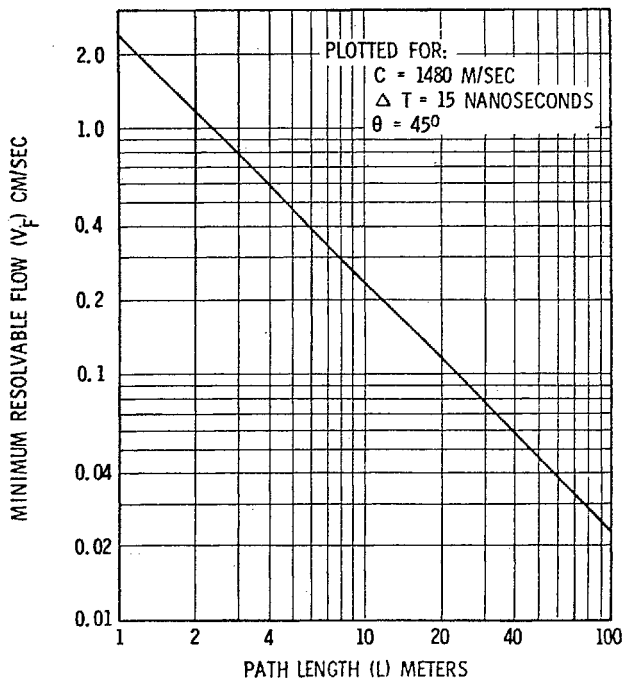


Figure 2. *Graph of minimum resolvable flow versus path length*

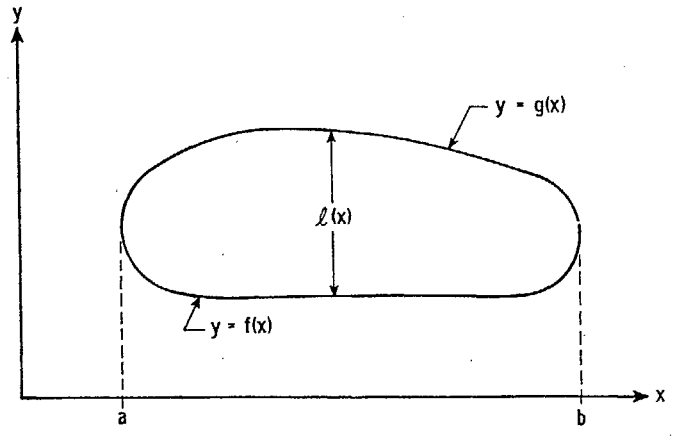


Figure 3. *Arbitrary conduit*

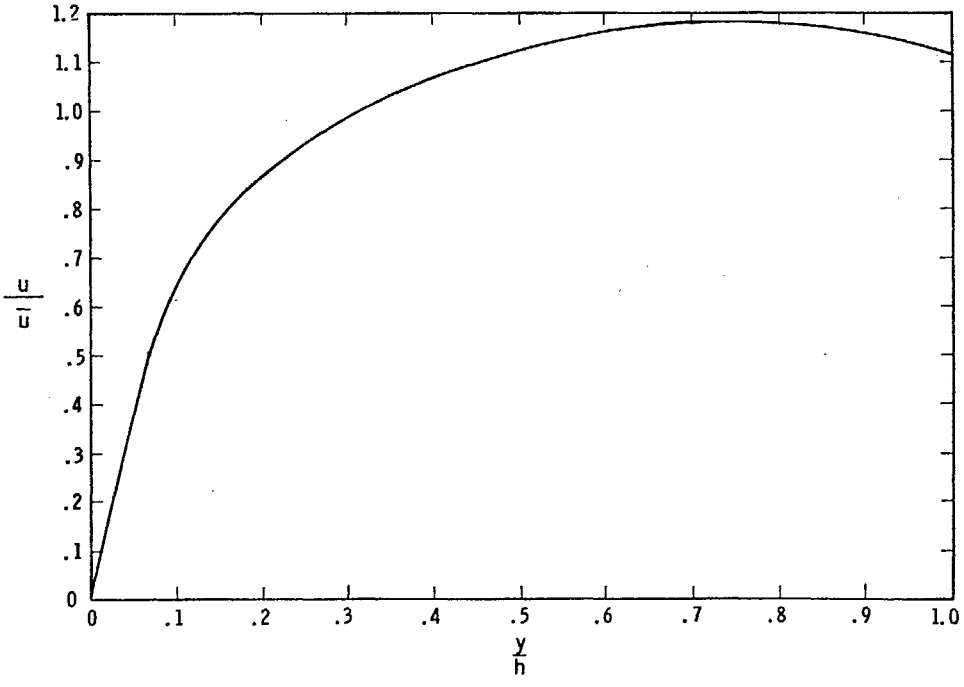


Figure 4. *Chicago Canal profile*

where L is the path length, C is the propagation velocity of the medium, V_F is the average spatial flow velocity, and θ the angle of the path with respect to the flow velocity. The LE flowmeter measures T_{AB} and the difference in transit times ($\Delta T = T_{BA} - T_{AB}$), and calculates flow velocity from the expression

$$V_F = \frac{L\,\Delta T}{2T_{AB}\,T_{BA}\cos\theta}. \tag{1}$$

As indicated by equation 1, this velocity calculation is a linear function of ΔT and is independent of the velocity of propagation in the medium; such dependence being accounted for by the measurement of T_{AB}. The number of paths implemented and the integration technique used changes somewhat the volumetric flow equations for different applications, but the flow velocity measurement is basic to all LE flowmeter systems.

Conditions of turbidity, sedimentation, and temperature gradients, which destroy the homogeneity of the medium, have resulted in the failure of several different acoustic techniques. The operational success of the LE flowmeter is, in a large part, due to the following characteristics of the system. First, the time measurements are made on the leading edge of the two received signals so that interference resulting from multipath propagation does not affect system performance. Second, a single pair of transducers is used for each path, with each used for both transmitting and receiving. Transmission occurs simultaneously from each transducer so that the ultrasonic signals travel identical paths in opposite directions during essentially the same time interval. Thus, there is no difference in the two path lengths and errors due to medium inhomogeneity are virtually eliminated. Third, the accuracy of the time measurement technique is not affected by the loss of occasional signals. Each transmit/receive cycle represents an independent velocity measurement.

EQUIPMENT DESCRIPTION

The basic LE flowmeter equipment configuration, as shown in Figure 1, consists of two transducers, a transmitter, two receivers, and a digital processor.

Acoustic signals are generated by simultaneously shock-exciting the two transducers, which causes them to ring at their resonant frequency. Since signal attenuation in the medium increases with frequency, transducer resonance is selected on the basis of measurement path length. Operating frequencies from 100 kHz to 1.6 MHz are generally used.

The received signals are amplified and the leading edges detected in balanced receivers to minimize difference in electronic time delays. The detector outputs provide the T and ΔT timing triggers for the digital processor.

The digital processor performs the functions of sequence control, receives signal validation, T and ΔT time measurement, arithmetic computation, display/peripheral equipment interfacing, and unit self test. To improve measurement accuracy and to smooth out the effect of hydraulic transients, multiple flow rate computations are made and averaged during each measurement interval. Typically, 64 independent time measurements are made on each of up to 4 paths during a measurement interval. Both T and ΔT are validated by determining that they are within an allowable range, and as many as 36 of the 64 samples can be lost without degrading system accuracy. Each flow measurement displayed is the average of the valid samples. An on-line self-test feature periodically checks equipment operation.

The basic flowmeter display provides decimal readouts of both flow rate and accumulated flow. However, interfacing electronics are provided to augment the system with a wide variety of peripheral equipment such as remote displays, telemetry links, digital or analogue printer, etc.

VELOCITY MEASUREMENT ACCURACY

Accuracy of the velocity calculation depends on the scaling of the path length L, path angle θ, and measurement of T_{AB} and ΔT.

Since all constants are implemented to 12 bit accuracy, the error due to L and θ is generally due to initial measurement error. Fortunately, by using standard surveying methods, it is possible to hold these errors to less than 0.1 per cent.

The flowmeter measures time by counting the number of clock cycles occurring during an interval of interest in 12 bit digital counters. Both T_{AB} and ΔT are quantized in this manner. However, since T is normally 100 times larger than ΔT, two different clock frequencies are used.

Selection of the clock frequency for T_{AB} quantization is determined primarily by the path length and the velocity of propagation in the medium. Since the path length is constant and the propagation velocity varies by less than 20 per cent for practical cases of interest, it is always possible to pick a frequency that ensures filling at least 11 bits of the 12 bit counter. This results in a T measurement which is accurate to more than 0.05 per cent.

Selection of the clock frequency for ΔT quantization is determined primarily by the path length, path angle, and the maximum flow velocity. When the propagation velocity is much greater than the flow velocity, it can be shown that ΔT is given approximately by the expression

$$\Delta T = \frac{2LV_F \cos\theta}{C^2} . \tag{2}$$

Therefore, unlike T, ΔT varies from 0 to some maximum value as a function of V_F. Error in measuring ΔT depends on the circuitry used in the digital processor, the number of measurements made in computing the average velocity, and the quantization frequency used. For the present system design, a ΔT error of 15 nanosec is achievable. Because of its limiting effect on measurement accuracy, the ΔT error has been associated with what is called the "minimum resolvable flow velocity". The minimum resolvable flow velocity is actually the velocity that would be calculated by using the maximum ΔT error ($\delta\Delta T$) and is given approximately by the expression

$$V_F\ (\text{Min}) = \frac{C^2 \delta\ \Delta T}{2L\cos\theta} . \tag{3}$$

For a given velocity of propagation and ΔT error, the minimum resolvable flow velocity is inversely proportional to path length. This relationship is plotted in Figure 2 for the nominal water propagation velocity of 1480 m/sec, a ΔT error of 15 nanosec, and a path angle of 45°. It is obvious from Figure 2 that long path lengths can reduce the minimum resolvable flow velocity to an insignificant value.

Considering all sources of error, the LE flowmeter provides flow velocity measurements to a guaranteed accuracy of 0.5 per cent of reading or equal to the minimum resolvable flow, whichever is greater.

DISCHARGE DETERMINATION

The property of an acoustic flowmeter of providing a spatial integral of velocity is an important and useful one in two respects. First, it provides very simply information that could only be approximated by a very large number of point measurements on a traverse, and, since the acoustic measurement is made virtually instantaneously, it can function in a transient situation where a lengthy and laborious traverse would be impossible. Secondly, the nature of the acoustic meter output is especially useful in that it leads naturally to a simple, but accurate, means of determining volumetric flows.

A conduit of arbitrary shape, which may be closed or open, enclosed by the boundary curves $y = f(x)$ and $y = g(x)$, with the flow in the direction perpendicular to the x-y plane, is shown in Figure 3. In the case of an open conduit, the free water surface is considered to form part of the conduit boundary. The volumetric flow rate in this conduit may be expressed as

$$Q = \int_a^b \int_{f(x)}^{g(x)} \cdot u(x, y)\, dy\, dx \tag{4}$$

where Q is the volume flow rate, and u is the axial velocity, Integration over y and substitution of $\ell(x)$ for $g(x) - f(x)$ yields

$$Q = \int_a^b \overline{u}(x)\ \ell(x)\, dx \tag{5}$$

where $\overline{u}$ is the average axial velocity along a line in a plane perpendicular to the conduit axis. The integrand is proportional to the value which would be read by an acoustic flowmeter positioned at x and oriented parallel to the y axis. This is true since the average path velocity times the path length ($V_p\, L$) may be related back geometrically to provide $\overline{u}l$ so long as axial variations of the flow profile can be neglected.

Since the flowmeter paths may be arranged across the conduit at will, it is possible to arrange them so that a numerical quadrature of equation 5 may be effected. This is done by weighting the readings of each of the acoustic paths and combining the resulting values.

This method of determining volumetric flow is a universal one; it does not require, as is the case with some methods, foreknowledge of, or assumptions about, the velocity profile for the determination of acoustic path locations. It can, therefore, work in cases where the flow has been distorted by bends, obstructions, etc. upstream of the meter. Further, symmetry of acoustic path locations renders the meter quite insensitive to many types of secondary, or swirling, flows.

The choice of a particular numerical integration scheme is not an easy one, for it is not possible to assert that a particular scheme will always perform better than any others in all distorted flow situations. For any reasonable number of acoustic paths, however, we may dismiss the various Newton-Cotes techniques (trapezoidal rule,

Simpson's rule, and higher-order counterparts) as being insufficiently accurate, as may be demonstrated readily by analysing their performance on the important case of a developed profile in a circular conduit. This leaves us to choose among the more sophisticated methods available. Suffice it to say that we have obtained good results by the use of Gauss' method and a variant thereof, Lobatto's method. The accuracy attainable depends upon the number of acoustic paths used and upon the extent to which the flow is distorted, but as few as three or four acoustic paths are capable of giving accuracies well under 1 per cent, uncalibrated for distorted as well as for fully developed turbulent profiles.

The gauging of open streams, when the water level is invariant, may be handled by the same method used for closed conduits. As in the closed conduit case, the conduit shape may be arbitrary, so that rectangular, trapezoidal or irregular channels are readily handled.

A modification of the basic method was developed which may be applied to open streams in which there is stage variation of modest amount. In addition to the acoustic transducers normally used, there is a need for a water level sensor. This method might be termed a hybrid system in that it uses a standard numerical quadrature system for part of the range and makes use of auxiliary measurements for the remainder. The technique is to use numerical quadrature over some portion of the flow which always remains below minimum stage. In this way, the proper location of paths in the numerical quadrature region is assured; and integration is always over a known fixed interval. The less the stage variation, the greater will be that portion of the flow which can be handled by the accurate numerical quadrature. For the remainder of the flow, one or more measurements may be made in the region between minimum stage and the top of the numerical integration region.

For a four-path open channel measurement system, it is recommended that three paths be used in the numerical integration region and a single auxiliary path be used to handle the remainder. A practical suggestion is that the upper limit of numerical integration and the auxiliary path be chosen such that the auxiliary path lies midway between the upper limit of numerical integration and the nominal stage depth, and that the auxiliary path be completely immersed at minimum stage.

Two examples may be given as illustrations of the performance of this system. The first is a logarithmic profile, which may be taken as representative of flow in a very wide channel. The profile is taken to be

$$\frac{u}{\bar{u}} = \frac{1 + 1.77\sqrt{C_f'}\ln y/h}{1 - 1.77\sqrt{C_f'}} \tag{6}$$

where $\bar{u}$ is equal to the volume flow rate divided by the conduit area, C_f' is the local friction coefficient, and h is the water depth. Suppose $C_f' = 0.0016$, for example, and that the stage variation is ± 10 per cent about nominal stage. Probably the most generally useful integration scheme is Gauss' method; it will be used here in its three-path form. In accord with the recommendations set forth above, Gaussian integration will be used in the region below $y/h = 0.75$, and the auxiliary path will be positioned at $y/h = 0.875$. Using the values given in Table 1, at nominal stage the flow is

$$Q_G + Q_{AUX} = 0.375 \left\{ \frac{5}{9} \left[\frac{u}{\bar{\bar{u}}} (1.7746 \times 0.375) + \frac{u}{\bar{\bar{u}}} (0.2254 \times 0.375) \right] + \right.$$

$$\left. + \frac{8}{9} \frac{u}{\bar{\bar{u}}} (0.375) \right\} + 0.25 \frac{u}{\bar{\bar{u}}} (0.875) = 1.0031, \text{ or } 0.31\% \text{ high}$$

where Q_G and Q_{AUX} are the volume flow rates in the Gaussian integration and auxiliary path regions respectively.

At 90 per cent stage,

$$Q_G + Q_{AUX} = \frac{0.375}{0.90} \left\{ \frac{5}{9} \left[\frac{u}{\bar{\bar{u}}} \left(1.7746 \times \frac{0.375}{0.90}\right) + \frac{u}{\bar{\bar{u}}} \left(0.2254 \times \frac{0.375}{0.90}\right) \right] + \right.$$

$$\left. + \frac{8}{9} \frac{u}{\bar{\bar{u}}} \left(\frac{0.375}{0.90}\right) \right\} + 0.1667 \frac{u}{\bar{\bar{u}}} \left(\frac{0.875}{0.90}\right) = 1.0041, \text{ or } 0.41\% \text{ high.}$$

At 110 per cent stage,

$$Q_G + Q_{AUX} = \frac{0.375}{1.10} \left\{ \frac{5}{9} \left[\frac{u}{\bar{\bar{u}}} \left(1.7746 \times \frac{0.375}{1.10}\right) + \frac{u}{\bar{\bar{u}}} \left(0.2254 \times \frac{0.375}{1.10}\right) \right] + \right.$$

$$\left. + \frac{8}{9} \frac{u}{\bar{\bar{u}}} \left(\frac{0.375}{1.10}\right) \right\} + 0.31818 \frac{u}{\bar{\bar{u}}} \left(\frac{0.875}{1.10}\right) = 1.0016, \text{ or } 0.16\% \text{ high.}$$

As a second example, consider the profile shown in Figure 4. This curve is the result of combining several measured curves in the Chicago Ship and Sanitary Canal. The nominal stage in this case is 7.3 m, with variation from 6.9 m to 8.8 m. Again, a three-path Gaussian quadrature is used and, in accord with the recommendation, it is used below $y/h = 5/6$. The auxiliary path is placed at $y/h = 11/12$. Without going into detail, by using Table 1 and reading values from Figure 4, we find that the flowmeter would read 0.8 per cent high at nominal stage, 0.3 per cent high at minimum stage and 0.7 per cent high at maximum stage under the assumption that the profile as a function of y/h does not vary with stage.

This method may also be checked with success against elliptic or parabolic laws used to represent velocity profiles in wide channels.

For narrower channels the velocity profiles are considerably altered due to secondary flow which carries high velocity fluid into the corners of the conduit. As

Table 1. *Gaussian abcissae and weights*

Number of paths	Abcissae, w_n	Weights, T_n
3	0	0.88888888
	± 0.77459667	0.55555555
4	± 0.33998104	0.65214515
	± 0.86113631	0.34785485

$$\int_a^b G(x)\, dx = \frac{b-a}{2} \sum_{n=1}^{N} G\left[\frac{b+a}{2} + \frac{b-a}{2} \cdot (w_n)\right] T_n$$

with circular conduits, the symmetry of acoustic path locations serves to effectively cancel out most of these effects. Care must be taken to ensure that the fluid velocity is at least reasonably close to being parallel with the channel. This dictates that the meter should not be installed in bends and that, in the case of main channels, the acoustic paths should be oriented with references to the main channel axis rather than the banks.

An LE flowmeter using several parallel paths can thus be used to determine volumetric flow rate in an open channel, to a high degree of accuracy, without calibration, even under conditions of distorted flow and variable water stage.

DISCUSSION

Oebius: Have you made any investigations on the influence of reflection from the bottom or the surface of shallow water on the accuracy of the determination of the velocity of the acoustic signal? Further, if there are two or more signals reaching the receiver, one shortly after the other, due to reflection, how great is the error?

Holmes: We have encountered this problem in certain situations and it is necessary to take steps in shallow waters, particularly in a wide but very shallow channel.

Oebius: Do you have values of the errors which could occur in such cases?

Holmes: No. In a very wide but shallow channel the difference in length between the reflected path and the direct path would not be very great because of the shallowness of the water.

Oebius: I do not agree, especially in the case of estuaries. Rivers, or some openings to the sea, are normally very very wide and very shallow with a deep track in the middle. In this case a reflection from the bottom is possible.

Holmes: If you worry about the reflection as compared with the true signal, the former signal is not read because the reception is time-gauged. The gate is only open for the short period of time when we anticipate the signal will appear, since we know approximately the speed of sound in the water and we know the path length.

Oebius: Another question: what about the influence of density and temperature on the velocity of the acoustic signal ?

Holmes: The density and temperature are automatically compensated for by the mechanization which is employed for determining velocities. We have two pieces of information. We have a time of flight for the upstream and a time of flight for the downstream signals. We also have two unknowns, the flow velocity and the speed of sound. Having computed the relationship on paper in order to find the velocity of sound, this result is then factored back into the system so that we are, in fact, automatically compensating on a propathed basis for the local speed of sound along that path.

Oebius: But at first you have to calibrate the speed of sound from one point to another?

Holmes: No. On any given path these two pieces of information are available. The device such as applied automatically, ensures that care is taken of the average speed of sound along that path. In the case of local high saline concentrations or influences from salinity, the change of the sound will be compensated for by the mechanization used for the determination of the average velocity along the path.

Oebius: What about the influence of suspended load and air bubbles on the use of acoustic signals for discharge determinations?

Holmes: If we can receive the signal, we can read the velocity.

Liu: It seems to me that on the basis of the equation presented in your paper that the travelling times T_{AB} and T_{BA} approach each other when the mean velocity of the flow is very small. Does it indicate that there is a lower limit of the transducer or of the device? What is the lowest limit you can measure?

Holmes: The error in the instrument, as far as measuring the velocity along the path, is in two parts really. One is the minimum resolvable flow due to the uncertainty of 15 or 10 nanosec, the other is at the higher flows due to errors in angle and path length.

Liu: What is the lowest velocity you have been able to measure?

Holmes: The lowest velocity was 0.02 m/sec at 1 m path length. In this range it is uncertain as to whether the flow errors are being measured.

Faber: How high is the background noise level? I am thinking about noise from shifting bedload in high mountain streams and of noises in canals caused by shipping.

Holmes: This is a function of the signalled noise ratio of the system. I cannot give you a figure on the noise ratio. We have not, to my knowledge, experienced any difficulty with it yet.

Gwinn: Are there any dangers to human beings from the use of this method?

Holmes: No.

Campbell: Can the system described be considered a prime measuring tool, which does not need to be calibrated, if a minimum of four path lengths are used in each of two intersecting directions?

Holmes: A number of different methods, including elevation in weigh tanks, Gibson-pressure momentum, and Alan salt velocity, have been checked. The general consensus has been that it is at least as accurate as any as those methods and it is more easily repeatable under normally identical conditions.

Hastings: I am a colleague of Mr. Whirlow who gave the paper and should like to make a few points in clarification.

1. The system has considerable operational experience dating back to 1964. A number of systems are in use in the U.S.A.

2. The Gaussian quadrature method has been used successfully in pipes and is now being applied to open channels for the determination of discharge using the LE flowmeter.

3. A number of features of the LE flowmeter make it immune to temperature and salinity changes and to reflections.

Bielenski: What was the largest width of rivers you have measured?

Holmes: The widest river in which we have operated so far is approximately 350 m.

A constant discharge siphon for flow measurement and control

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Abstract. A simple, inexpensive device consisting of a floating siphon system can be used both to discharge water from a container at a constant rate independent of the stage in the container and, simultaneously, to measure the variable rate of flow discharged into the container, provided the downstream water stage is low enough for the siphon to discharge water freely into the atmosphere. The device under investigation consists of hollow polyethylene boxes as the float and bent polyethylene tubes embedded in the boxes as the siphons. The total constant discharge of the siphon system can be varied by controlling the number of the siphons in operation and the siphon head. The float is connected to a stage recorder to obtain a discharge record.

SIPHON À DÉBIT CONSTANT POUR LA MESURE ET LE CONTROLE DU DÉBIT

Résumé. Un appareil simple et bon marché qui consiste en un système flotteur-siphon peut être utilisé pour vider l'eau d'un container, à un débit constant indépendant de la hauteur d'eau dans le container, et simultanément pour mesurer le débit variable déversé dans le container, à condition que le niveau de l'eau en aval soit suffisamment bas pour que le siphon puisse débiter l'eau librement dans l'atmosphère. Cet appareil, décrit en détail, consiste en boîtes creuses de polyéthylène (c'est le flotteur) et en tubes de polyéthylène courbés et encastrés dans les boîtes (ce sont les siphons). On peut faire varier le débit constant total du système flotteur-siphon en contrôlant le nombre de siphons en fonctionnement et la charge du siphon. Le flotteur est relié à un limnigraphe enregistrant le débit.

SIFÓN DE CAUDAL CONSTANTE PARA MEDICIÓN Y CONTROL DE CAUDAL

Resumen. Se puede utilizar un dispositivo sencillo y barato de sifón flotante que descarga agua en un depósito a una velocidad constante con independencia del nivel del depósito y para medir simultáneamente el valor de caudal descargado, con tal de que el nivel del agua, aguas abajo, sea suficientemente bajo para que el sifón pueda desaguar libremente. Este dispositivo que se está investigando, consiste en cajas de polietileno huecas como flotador y tubos de polietileno curvados introducidos en las cajas como sifones. El caudal constante del sistema de sifón se puede variar controlando el número de sifones en funcionamiento y la altura del sifón. El flotador se conecta a un registrador de nivel, a fin de obtener el registro de caudal.

СИФОН ПОСТОЯННОГО РАСХОДА ДЛЯ ИЗМЕРЕНИЯ И КОНТРОЛЯ СТОКА

Резюме. Простое, недорогое устройство в виде системы плавающего сифона может быть использовано для выпуска воды из контейнера с мостоянным расходом независимо от уровня воды контейнере и одновременно может измерять переменный расход, поступающий в этот контейнер; при этом уровень воды в нижнем течении должен быть достаточно низким с тем, чтобы бифон мог изливать воду свободно в атмосферу. Это устройство состоит из полых полиэтиленовых ящиков, причем поплавок и изогнутые полиэтиленовые трубы вделываются в эти ящики, образуя сифоны. Полный постоянный расход системы сифонов может регулироваться путем измене-

ния числа сифонов, находящихся в действии, и напора сифонов. Поплавок соединяется с самописцем уровня, чтобы получить запись измеренного расхода.

INTRODUCTION

In hydraulic engineering works it is often necessary to discharge water from a reservoir or container at a constant rate. One example is the ancient water clock. Recently a device which can discharge water at a constant rate from a container, independent of the stage, has been developed at the University of Illinois as a component of the Watershed Experimentation System (WES) for the purpose of flow measurement and control in connexion with the watershed hydraulics research.

The details of WES have been reported elsewhere (Chow, 1967, 1968). In this experimental system, run-off from the laboratory watershed is collected in a storage tank consisting of three compartments. The run-off collected in the central compartment is then pumped out, while the stage in the compartment is measured by a pair of sonar depth gauges at time intervals of 4 or 5 sec. The run-off rate from the watershed is computed on the basis of the continuity principle applied to the central compartment. As the watershed run-off and the pumping rate are non-uniform and cover a wide range from zero to 0.02 m^3 per sec, the accuracy of this method of run-off measurement is limited. This accuracy, however, can be improved greatly if water is removed from the central compartment at a known constant rate instead of direct pumping, thus minimizing the disturbance in the compartment.

The proposed device is a floating siphon system which siphons water from the central compartment into the two side compartments. The water surface in the two side compartments is kept sufficiently low so that the siphons can discharge water freely into the atmosphere. The floating siphon system has the advantage that for different experiments the constant flow rate in each experiment can be changed by varying the head and the number of siphons in operation. In addition, by connecting the float of the system to a stage recorder the water surface elevation in the central compartment of the tank can be recorded continuously in complement to the discrete sonar readings. In the design and operation of the siphon system, the following factors should be considered:

1. The system must be sufficiently accurate and sensitive in both discharge control and stage measurement;
2. The system must be able to withstand small disturbances such as surface waves;
3. The system must be relative simple and easy to operate;
4. The system must be inexpensive and applicable over a wide range of operating conditions.

THEORETICAL ANALYSIS

Flow theory. The theoretical basis for the floating siphon system is quite simple. As shown schematically in Figure 1, the float is supposed to follow the change of the water surface in the tank so that the difference in head, H, between the water surface and the outlet section of the siphon is constant. By using the one-dimensional approach, if the rate of change of water surface in the tank is not rapid, the Bernoulli equation yields

$$H = \left(f\frac{L}{D} + K_1 + K_e + K_b + K_m \right) \frac{V^2}{2g} \tag{1}$$

in which f is the Weisbach resistance coefficient; V, L, and D are respectively the mean velocity of flow, total length, and diameter of the siphon tube; K_l, K_e, K_b, K_m are the loss coefficients due to exit, entrance, bends, and other minor losses such as connexions, respectively; and g is the gravitational acceleration.

For a steady, uniform flow the value of f can be obtained from the Moody diagram. For the condition under consideration, the exit loss coefficient $K_l = 1.0$ and the entrance loss coefficient $K_e = 0.5$ or slightly higher (Rouse 1950). The bend loss coefficient K_b usually ranges from 0.1 to 0.5 for 90° bends; and it is a function of the Reynolds number, relative roughness of the tube, and the ratio of bend centre-line radius to tube diameter.

If multiple siphon tubes are mounted closely to the float, equation 1 is still applicable. The loss coefficients for each tube are practically unaffected by the presence of the other tubes except K_e, which would be slightly modified according to the arrangement of the tubes at the entrance.

Response of siphon system to change of water surface elevation. The sensitivity of the response to a change of water surface elevation, and hence the accuracy of the siphon system, depends on the mass of the system, m, the waterline cross-sectional area, A, and the frictional resistance to motion, F_r. If the siphon system is connected to a stage recorder by a cable, there is an additional force due to cable tension T acting on the system. For a small change of elevation of water surface in the tank, dz, Newton's second law gives

$$\gamma A \frac{dz}{dt} \Delta t - F_r + \Delta T = ma \tag{2}$$

in which γ is the specific weight of water, a is the acceleration of the siphon system, Δt is the small increment of time corresponding to the change dz, and $\Delta T = T - T_e$ where T_e is the equilibrium cable tension when the float is at rest. The sign of T is the same as z, being positive upward. Hence from equation 2,

$$\Delta z = \frac{ma + F_r - \Delta T}{\gamma A}. \tag{3}$$

Note that $F_r > 0$ if the float is rising with a rising water surface having a positive Δz and $F_r < 0$ if the float is falling with a negative Δz. To avoid an undesirable inertial effect of the siphon system, the product ma should be kept small. Thus, the siphon system should be as light as possible.

Assuming that, at rest, the tension in the cable on the float side of the stage recorder pulley is counterbalanced by a weight at the other side of the pulley, i.e., an Atwood machine system, four different cases can be considered for equation 3:

1. If both the siphon system and the counter weight are in uniform motion, then $a = 0$ and $da/dt = 0$. Hence, equation 3 gives

$$\Delta z = \frac{F_r - \Delta T}{\gamma A} \tag{4}$$

in which $\Delta T = -F_r'$ for a rising float with $F_r > 0$, and $\Delta T = F_r'$ for a falling float with $F_r < 0$. Here $F_r' > 0$ is the dynamic frictional resistance of the pulley-counter weight system, i.e. additional resistance due to the recorder. If the float system is not connected to a stage recorder, then $\Delta T = 0$ and $\Delta z = F_r / \gamma A$. Thus, it is clear that by connecting the float to a stage recorder the accuracy is decreased by an amount equal to $F_r' / \gamma A$.

2. For the case of incipient motion of the siphon system, $a = 0$ and $da/dt \neq 0$. Again, from equation 3, equation 4 is obtained. However, in this case F_r' and F_r are equal to the maximum static frictional resistances due to the recorder and the float respectively, which are greater than the corresponding dynamic frictional resistances for the preceding case of uniform motion. It is obvious that for both cases the reduction in friction due to the stage recorder would improve the accuracy of measurement by using the float to indicate the stage.
3. If the float accelerates with a frictionless pulley-counter weight system, equation 3 applies. The sign of ΔT is opposite to a, i.e., for $a > 0$, $\Delta T < 0$. For an upward motion the acceleration ($a > 0$) implies a larger Δz since $F_r > 0$, and hence for a given F_r accuracy is decreased, whereas a deceleration implies an improved accuracy. However, for a downward motion with $F_r < 0$, acceleration would improve the accuracy whereas deceleration would worsen it.
4. If the float is accelerating and connected to a frictional stage recording system, again equation 3 applies. The frictional resistance of the recorder system produces more error than that for case 3, and the worst conditions are the case of a downward motion of the float system with a rapid deceleration and the case of an incipient upward motion. It should be noted that in this case the error in stage or discharge measurement is non-uniform, being a function of the acceleration as well as the velocity of the siphon system. Fortunately, in practice the acceleration rate is usually small so that the variation in error due to different conditions of motion is relatively small.

From the above considerations it is obvious that, for a desired accuracy for stage measurement with small Δz, it is necessary to either increase the waterline area of the float or to minimize the friction guiding the vertical motion of the floating siphon system.

Accuracy of discharge control. The discharge from the floating siphon system can be evaulated by using equation 1. For each siphon the discharge, irrespective of the elevation of the water surface in the tank, is,

$$Q = \frac{A_s}{\sqrt{C}} \sqrt{2gH} \tag{5}$$

in which A_s is the cross-sectional area of the siphon tube and

$$C = f \frac{L}{D} + K_1 + K_e + K_b + K_m. \tag{6}$$

As a result of the mass and frictional resistance to motion of the siphon system, there may be a lag in the response of the system to the motion of the water surface. If the deviation of the actual head from the equilibrium siphon head is Δz, then the modified discharge is

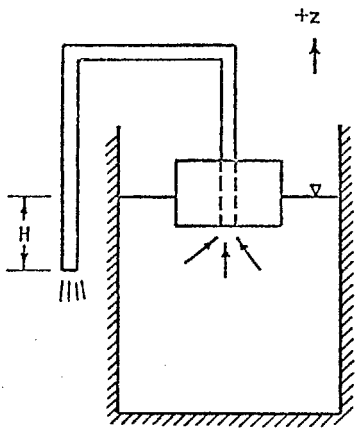


Figure 1. *Schematic diagram of a simple floating siphon system*

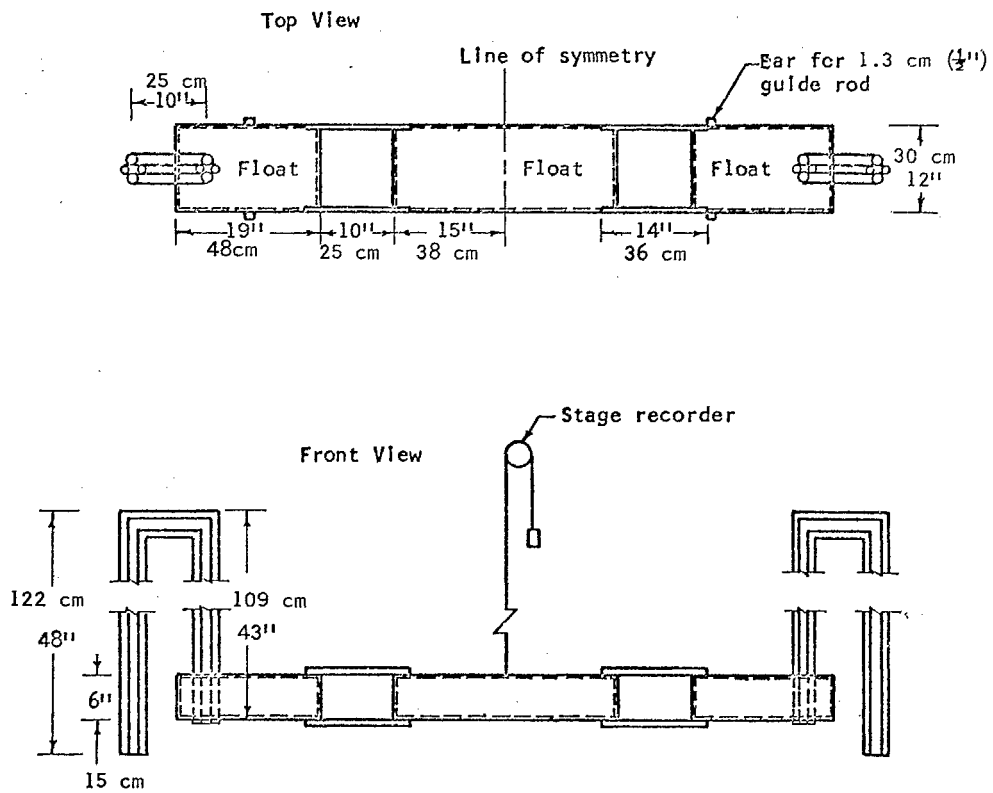


Figure 2. *The floating siphon system*

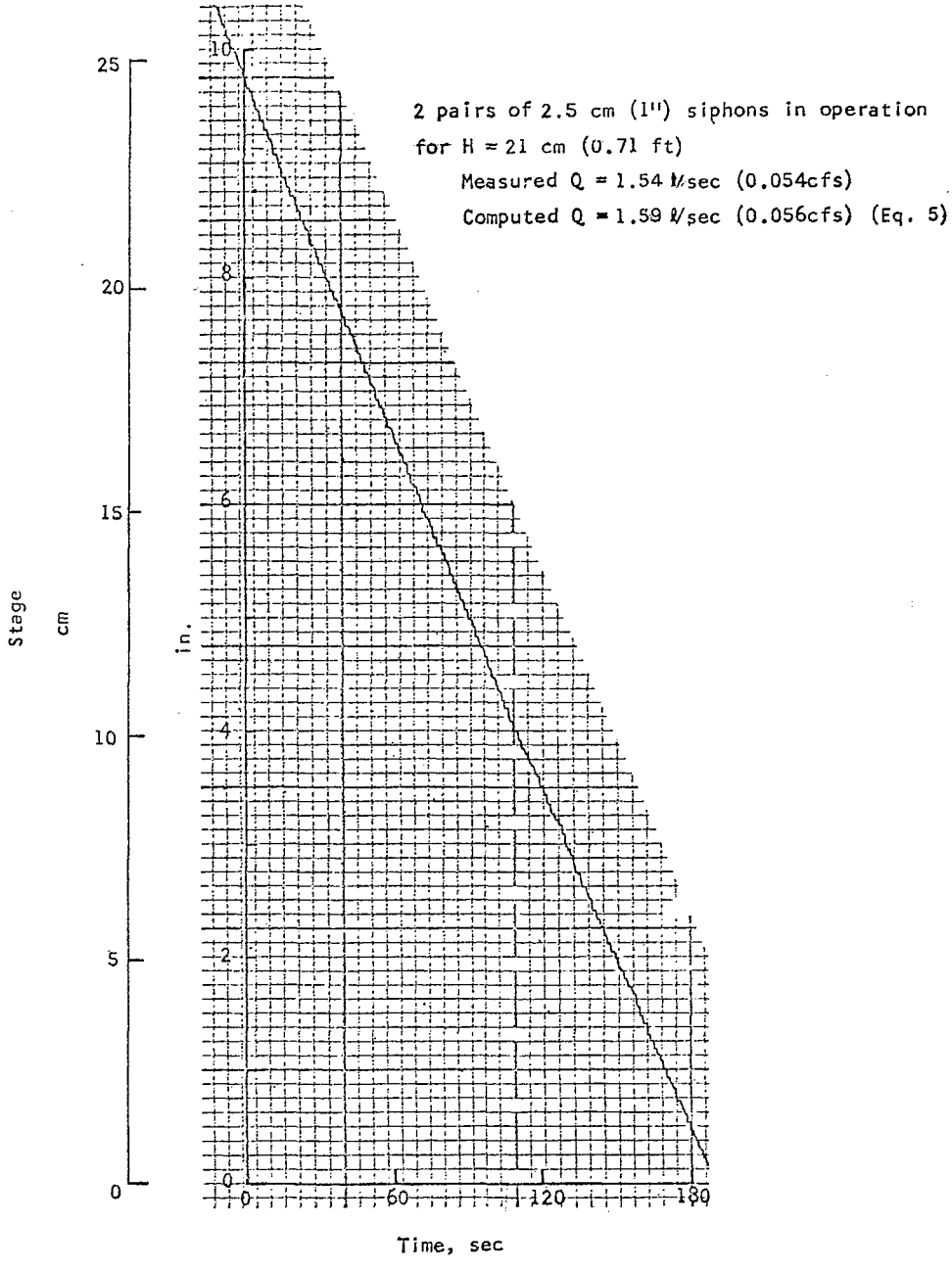


Figure 3. *A calibration curve*

$$Q' = \frac{A_s}{\sqrt{C'}} \sqrt{2g(h + \Delta z)} \tag{7}$$

where Δz is positive if the water surface is above the equilibrium waterline of the float. Since the difference between Q and Q' is small, C and C' are approximately equal. Hence, the relative error of discharge due to Δz is

$$\frac{Q' - Q}{Q} = \frac{\sqrt{H + \Delta z} - \sqrt{H}}{\sqrt{H}}$$

$$= \frac{1}{2}\frac{\Delta z}{H} - \frac{1}{8}\left(\frac{\Delta z}{H}\right)^2 + \frac{1}{16}\left(\frac{\Delta z}{H}\right)^3 - \cdots \tag{8}$$

or, approximately,

$$\frac{Q' - Q}{Q} = \frac{1}{2}\frac{\Delta z}{H}. \tag{9}$$

EXPERIMENTS

Apparatus. The floating siphon system for testing, as shown in Figure 2, consisted of one centre float and two identical side floats connected as a unit. The floats are hollow, rectangular, polyethylene boxes. The siphons were constructed from two 2.5 cm and two 3.8 cm inverted polyethylene U-tubes embedded into each of the two side floats. The dimensions of the siphon system were selected in such a manner that they would fit the existing experimental setup and the requirements for WES. Polyethylene was used to construct the floats and siphons primarily because of its relatively low specific gravity (0.92) and low water absorption (< 0.01 per cent) although it is relatively flexible and has low tensile strength. When the weight consideration is not critical and greater strength is required Plexiglas may be used.

The centre float was used to increase the waterline cross-sectional area A in order to improve the accuracy and sensitivity as discussed in the preceding section. A cable attached to the centre float was connected to the pulley of a 51 cm Leupold & Stevens recorder to record the stage of water in the tank. In order to restrict the system to vertical motion, four guide ears were connected to the float system, with each ear riding on a fixed low friction 1.3 cm diameter Plexiglas rod.

The outlet of the siphons ranged from 19.5 cm to 24.5 cm below the equilibrium waterline of the float at rest, depending on the number of siphons in operation. Additional siphon head could be obtained by telescoping additional lengths of appropriate size polyethylene tubes to the end of the siphons. At the highest point of each siphon tube a 0.6 cm diameter vinyl tubing was used as a bleeding passage to permit priming. When priming was completed the passage was closed by means of a pinch clamp.

Calibration. First the response of the system to changes in water surface elevation was investigated. By pumping water into or out of the tank at a small constant rate with no siphon action, the operating condition is approximately that of case 2 in "Theoretical Analysis". With the water surface rising or falling at the rate of

approximately 0.03 cm/sec, the float system was found to respond to a change $\Delta z = 0.12$ cm.

The siphon system was then calibrated for the case of uniform motion with the siphons in operation, which is case 1 in "Theoretical Analysis".The siphons were run in pairs, symmetrical with respect to the axis of symmetry of the system, for maintaining dynamic balance. It was found that the response to change of water surface, Δz, was slightly improved over the case with no siphon flow. A typical calibration curve is shown in Figure 3 for two pairs of 2.5 cm siphons running under a head of 21 cm and discharge of 1.54 *l*/sec. Tests were also made with the floating siphon system accelerating, as in case 4. No appreciable change in response, Δz, was observed.

The constancy of the siphon discharge can be seen from the slope of the experimental stage-time curves (Figure 3). Within the measurable range, the discharge thus measured was constant for each experiment. This high accuracy in the discharge control is to be expected, as equation 9 shows that the theoretical error is $\Delta z/2H$ or approximately 1/300.

After the siphon system is calibrated the run-off from the laboratory watershed can be computed. At any instant the run-off rate is equal to the sum of the siphon discharge and the time rate of change of the volume of water in the tank. The siphon discharge is known from the number of siphons in operation and the head for each. The volume of water in the tank is obtained from stage measurements and a known stage-volume relationship for the tank.

CONCLUDING REMARKS

It has been shown theoretically and verified experimentally that a system of siphons mounted on floats can be used as a flow control device for constant discharge or stage measurement purposes. The error involved in discharge due to the inertia and friction of the floating siphon system is approximately $\Delta z/2H$, as shown in equation 9. This error is small in most applications, since $\Delta z/2H$ can easily be made smaller than 0.1. If the siphon system is also used for accurate stage recording, requiring small Δz, special attention should be given to the reduction in the mass and the mechanical resistance to motion of the system, as well as to the increase of the waterline cross-sectional area of the floats.

In engineering practice, a floating siphon system may be built inexpensively, particularly if the stage measurement is not a requirement. There are numerous engineering applications for a constant flow siphon system. These include diversion of water from a river or canal, selective withdrawal at a constant flow rate from a reservoir, constant discharge to ensure fulfilment of downstream water rights requirements or irrigation water demand, and selective diversion at constant flow rate of storm sewer run-off from a temporary storage facility for sewage treatment or direct disposal purposes.

ACKNOWLEDGMENTS

The research presented in this paper is a part of a continuous investigation on watershed hydraulics in the Department of Civil Engineering of the University of Illinois at Urbana-Champaign, Illinois, U.S.A. This research is financially supported

by the United States National Science Foundation under Grant GK-11292, "Hydrodynamics of Watershed Flow". The construction and calibration of the floating siphon system apparatus was accomplished by Mr. Yung-Yuan Shen, a Graduate Research Assistant.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Chow, V. T. 1967. Laboratory study of watershed hydrology. *Proc. Internat. Hydrology Symposium,* vol. 1, p. 194-202. Ft. Collins, Colorado, U.S.A.

Chow, V. T. 1968. R and D of a watershed Experimentation System. *Jets J.,* vol. 15, no. 6, p. 11-13.

Rouse, H. (ed.) 1950. *Engineering in Hydraulics,* New-York, Wiley. 415 p.

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III Position fixing
Détermination du point

The development and performance of a new high accuracy hydrographic Tellurometer model MRB 201

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Tellurometer Ltd., Chessington, Surrey, United Kingdom

Abstract. This paper discusses the development of a new high accuracy (± 1 m) hydrographic Tellurometer. The various techniques and special features of the equipment are discussed and the results of field trials conducted with the prototype are included.

UTILISATION ET PERFORMANCES D'UN NOUVEAU TELLUROMETRE HYDROGRAPHIQUE DE HAUTE PRECISION, MODELE MRB 201

Résumé. Cet article traite du développement d'un telluromètre hydrographique de grande précision (± 1 mètre). L'auteur décrit les différentes techniques et les particularités du matériel et il donne les résultats des essais pratiques effectués avec le prototype.

DESARROLLO Y UTILIZACIÓN DEL NUEVO TELURÓMETRO DE ALTA PRECISIÓN, MODELO MBB 201

Resumen. Se explica el desarrollo de un nuevo telurómetro hidrográfico de alta precisión (± 1 m). Se discuten las diversas técnicas y características especiales del equipo y se incluyen los resultados de ensayos en campo realizados con el prototipo.

УСОВЕРШЕНСТВОВАНИЕ И ЭКСПЛУАТАЦИЯ НОВОГО ВЫСОКОТОЧНОГО ДАЛЬНОМЕРА ДЛЯ ГИДРОГРАФИЧЕСКИХ РАБОТ, МОДЕЛЬ MRB 201

Резюме. В статье рассматривается усовершенствование нового высокой точности (± 1 м) дальномера. Описаны различные методы и особые характеристики оборудования, а также результаты полевых опытов с этим прибором.

Not long after the introduction of the first Tellurometer (which very quickly set new standards of speed and accuracy for survey distance measurements), inquiries were received as to the possibility of using the instrument for hydrographic purposes. Unfortunately hydrographic requirements are not compatible with land survey ideals, mainly because the instrument would be used in a dynamic mode—this means that the high resolution desired on land becomes somewhat of an embarrassment for a rapidly changing distance. These drawbacks were recognized and after some experiment and development, MRB2 was put on the market in 1960. This instrument had a direct readout in metres with resolution (1 m) reduced compared with the normal Tellurometer. This instrument performed very well and has been in continuous production ever since. However, it was decided at the beginning of 1969 to replace it with a more up-to-date model—MRB201— the main reasons being that MRB2 was a valve instrument and as such had a rather heavy battery drain and that it had a purely

manual readout, there being no provision for binary coded decimal output or any form of remote readout. This last point had been a drawback right from the start.

In developing the new model the following points were borne in mind:

1. A range of a least 50 km;
2. Readout display must be digital;
3. Binary coded decimal output so that the instrument could be used with peripheral equipment such as tape punches, plotting computers, etc;
4. Fully transistorized circuitry to reduce power consumption;
5. Must be as fully automatic in operation as could conveniently be arranged, bearing in mind that the MRB201 was to be in the modest price category;
6. Also, to keep the price down and to keep troubles to a minimum, as much as possible if existing and proved equipment was to be used;
7. Resolution to be such that accurate static measurements could be made, but at the same time dynamic readings would not change too rapidly to be read by eye if necessary.

To implement points (6) and (4) an established model (MRA3) Tellurometer was used as the basic instrument. This instrument, however, had a carrier of 10 GHz and a pattern frequency of 7.5 MHz giving a resolution of 1 cm, which was counter to points (1) and (7). The carrier frequency was, therefore, changed to 3 GHz and the patterns to 1.5 MHz—these were both taken from an earlier hydrographic instrument, the MRB2. Point (5) was dealt with by designing a completely new pattern oscillator controlled by a system of tones in the 39 kHz to 76 kHz band—this system has proved very satisfactory, allowing fully automatic measurement to be carried out controlled wholly from the master unit.

The remaining points are all related to the readout itself. There are in fact two different plug-in readout units available, one being the standard MRA3 type which uses a conventional phase resolver system to give a resolution of 0.1 m. This unit will be offered for use with all remote instruments to enable these instruments to be used to measure their own baseline. The second readout unit – the digital range integrator (DRI) – is a far more elaborate device designed specifically for dynamic measurements. It also has a resolution of 0.1 m, but the last digit is normally ignored whilst in rapid motion.

DESCRIPTION OF THE SYSTEM

This instrument is intended to be used in all ways in which its predecessor, the MRB2, could be used. Briefly, these were: (a) two range system; (b) range angle system; and (c) airborne control system.

Of these, (a) is the usual system used for most offshore surveys. Here two pairs of instruments are set up, two instruments, switched to remote function, being established at known points on the shore and the remaining two as masters installed on board the survey vessel. Here the MRB201 has a clear superiority over the earlier MRB2 in that the shore instruments, normally with the standard dial readout, can be used to measure their own baseline. The earlier instrument had to have its master unit unshipped from the survey vessel, taken on shore with appropriate batteries, used to measure the line and then reinstalled; alternatively a pair of additional instruments – usually MRA2's, MRA3 or MRA101 – had to be provided for this sole purpose.

The master instruments would be fitted with the DRI, which gives continuous range information in digital form once contact with the shore stations has been

established; this DRI is described later in this paper. The DRI provides a binary coded decimal output which can be fed straight into a computer driving an *X-Y* plotter, thus making continuous position plotting possible.

The range/angle system involves only one pair of instruments plus a survey theodolite. Here the master is more conveniently installed on shore along with the theodolite. Measuring consists of taking simultaneous range and angle measurements. This mode of measurement is convenient in work where motion is slow and boats have to be placed at predetermined positions—this covers dredging, bottom sampling and underwater construction. Here the fact that the remote unit is on the vessel and the size of the instrument being such as to allow very compact installation, makes it possible to have a relatively small boat which can work in shallow water. This is also facilitated by the fact that the remote unit is completely automatic, thus all control can be from the shore station.

The airborne control system is an elaboration of (b). Here a remote unit is fitted into a specially equipped helicopter, while the master unit is stationed at a known point together with a theodolite. This system is of great use in mountainous and inaccessible regions to give rapid ground control for aerial photography. In use, the helicopter stations itself above the unknown point (it is held stable by its special autopilot) by means of its optical hoversight and measures its height above the point by a special plumb line. Now the master operator takes a reading of range to the remote antenna slung below the helicopter while the theodolite is used to take elevation and azimuth to a high intensity flashing beacon mounted above the antenna on the helicopter.

For use in systems (b) and (c) it would be possible to use the conventional dial readout instead of the DRI (with some economy in basic investment) as movement is slow or non-existent.

It will be seen that with the switched master/remote function coupled with a choice of readout plug-ins this instrument has very great flexibility of use; additionally it has a big advantage in having considerably lower current consumption. When measurements on the move are being made, the normal choice of readout would be the DRI, which is described below.

The DRI was designed to have what might be termed the electrical analogue of the flywheel effect by making it sense the rate of change of distance and lock to this. In other words, if for any reason signal is lost for up to 20 or 30 sec, the instrument will go on integrating range at the rate at which it was going immediately before the loss of signal. This rate sensing system has the added advantage that it provides averaging for rolling and pitching and other ship movements, as it tends to follow the average rate rather than the actual movements of the ship. As opposed to ordinary types of averaging, there is very little time lag, approximately 1 millisec. Operation is extremely simple—at the the beginning of the exercise the instrument is used to read out the full range using all coarse patterns. These readings are then entered into the DRI by pressing the "READ IN" button. After the ship has got under way, the instrument is returned to its main pattern, and the rate circuit pulls into the average rate and integrates the distance as the range varies.

If at any time the operator wishes to check the accumulated readings it can be done, without stopping the vessel, by means of the comparison meter built into the unit.

The instrument as a whole can be connected to a computer and set to its "Auto Master" mode in which case the computer has direct control of patterns. The DRI

need not be used in its rate sending mode under these circumstances, as the computer can produce a good average itself.

The instrument has been designed to work up to a maximum rate of 30 knots (15.4 m per sec). To enable the operator to take down readings manually at maximum speed, a "HOLD" button has been provided which holds the reading on the Nixie tube readout for as long as it is desired. The display updates as soon as the "HOLD" button is released.

This is a description of how the instrument operates. Although both instruments are identical and merely switched to a master or remote function, for the purposes of this description I will treat the master as if it is a different instrument to the remote.

Master function

Basically the instrument embodies four continuously running oscillators feeding into four gates which are controlled from a tone generator. The selected pattern feeds through a pattern amplifier and then into a klystron tuning unit, generating a carrier of 3000 MHz, where it is frequency modulated onto this carrier. In operation, select say the master A+ pattern (that is the main pattern). This would not switch the gates directly; in actual fact it switches a tone generator which produces a 61.37 kHz tone. This tone is detected by a detector built into the tone generator producing a d.c. output fed to the appropriate pattern gate which opens and allows the pattern frequency, in this case 1,498,470 Hz, to modulate the klystron. In addition, the 61.37 kHz tone is also modulated onto the 3,000 MHz carrier. This 3,000 MHz with its two associated modulations is fed to a dipole which is set in the parabolic dish; whence it is radiated, to be picked up by the remote.

The receiving part of the instrument consists of a mixer, a 33 MHz amplifier which feeds into a 33 MHz limiter discriminator and an a.m. detector. The a.m. detector is followed by a 1 kHz amplifier/limiter which feeds into one of the optional readouts. The limiter/discriminator feeds into a 100 kHz filter/limiter which then drives a 100 kHz discriminator and leads into a 1 kHz amplifier and from there also into the readout. The receiver picks up 3,033 MHz from the remote station and mixes a small portion of the master 3,000 MHz to give a difference frequency of 33 MHz.

Remote function

The remote receiver, which consists of a line-up identical to the master except that the output from the 33 MHz limiter discriminator, which in this case will be 61.37 kHz, is fed into a tone detector which produces a voltage out of the appropriate output. This voltage is fed to an identical set of pattern gates, turning on the A+ gate. The transmit side comprises five oscillators feeding into five gates which drive a pattern amplifier and then into the klystron tuning unit. In this case the carrier frequency would be 3,033 MHz. Some of this 3,033 MHz is bled back into the receive dipole where it mixes with the originally received 3,000 MHz to give a 33 MHz intermediate frequency plus a 1 kHz a.m. (which is the result of the master and the remote pattern frequencies beating together). In addition there will be a 61.37 kHz f.m. originally received from the master. This 1 kHz a.m. is detected by the AM detector, goes into a 1 kHz amplifier limiter and is then used to modulate a 100 kHz oscillator (f.m.). This 100 kHz, now frequency modulated by 1 kHz, is also fed into the klystron tuning unit. The remote in actual fact transmits 3033 MHz plus 1,497,470 Hz f.m. plus a complex modulation consisting of 100 kHz subcarrier

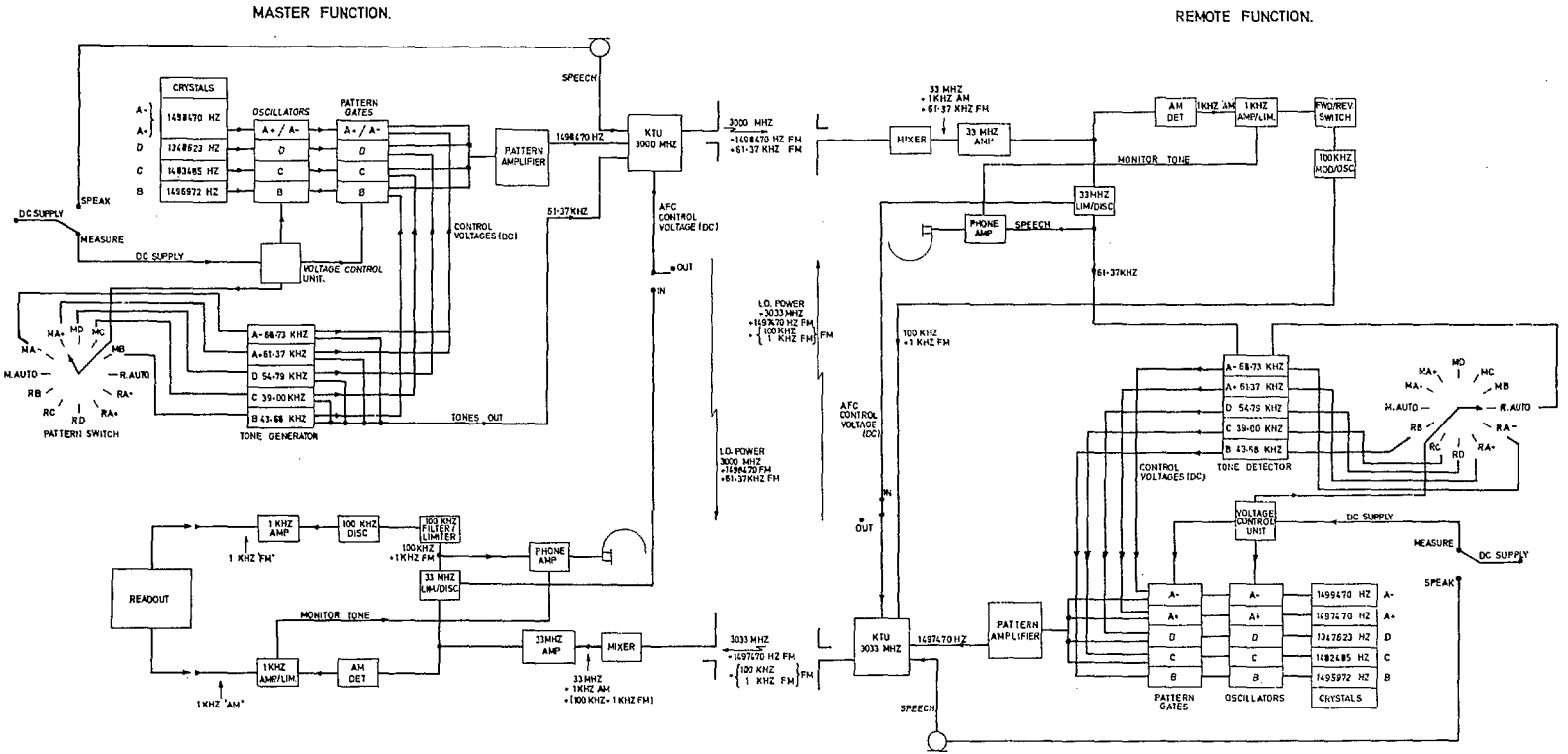


Figure 1 – *MRB 201 block diagram*

Table 1. *Simplified modulations*

	Master	Remote
Transmitted :	3 000 MHz + 1498470 Hz f.m. + 61.37 kHz f.m.	3 033 MHz + 1497470 Hz f.m. + (100 kHz) f.m. (+ 1 kHz f.m.)
After mixer:	33 MHz + 1 kHz a.m. + (100 kHz) f.m. (+ kHz f.m.)	33 MHz + 1 kHz a.m. + 61.37 kHz f.m.
Out of 33 MHz discriminator:	100 kHz	61.37 kHz
Out of AM detector:	1 kHz (Ref)	1 kHz
Out of 100 kHz discriminator:	1 kHz	

which has been frequency modulated by 1 kHz back to the master. The simplified modulations are shown in Table 1.

Looking at the block diagram (Fig. 1) it will be seen that in the master function there are four oscillators although there are five tones, the reason being that the additional tone is required to control the remote A-switching. The tone generator is, in fact, the identical unit to the tone detector, the pattern switch controlling which function is to be performed. In the same way the pattern gates are identical on both instruments. All instruments carry a full set of nine crystals, each with its associated oscillator; in other words, the four master and five remote patterns (in master function only the master oscillators, and in remote function only the remote oscillators are energized). Because of the way these pattern gates and their tone generators are arranged, extremely rapid switching is obtained.

It will be seen that all the remote patterns can also be manually selected—this is mainly a convenience in servicing and setting up the crystals. The only unexplained position on the pattern switch is the master Auto. In this position the pattern gates are connected to an external plug, none of the gates will be activated until the appropriate voltages are injected into this plug—this is provided so that the instrument can be controlled from a master programming unit which could provide a full pattern print out on demand. This master programmer could also be controlling automatic echo soundings or a plotter or any other form of peripheral gear such as a computer printout.

RESULTS

To date no dynamic tests have been performed, but a large number of static tests of varying ranges have been carried out. (A prototype of the DRI has been bench tested using a simulator; the results were very promising). These tests range from a minimum of just over 100 m to just under 70 km. Representative results are included in Appendix 1. It will be seen that even at a range of 66 km signal strength is still considerable, the AGC reading on the weaker of the two instruments being 17 μA which corresponds to a signal of 8 1/2 μV at the I.F. input. Additional loss of signal was introduced by swinging the units out of line and ultimately it was found just possible to take reasonable readings at an indicated AGC of 3 μA corresponding to a

signal of 4 μV. This means that the signal at this range was a minimum of 6.5 Db above absolute limit. This absolute lower limit was defined for purposes of this test as that level at which noise was sufficient to cause the null meter to oscillate $\pm$ 10 μA. Naturally this lower limit will vary depending on the line and the weather; during this test the weather was hot and rather windy and this line is somewhat noisier than average as it passes over hot sand alternating with bush which causes turbulence, i.e. local variations of refractive index, in the air above.

It will be seen from the results that the pattern breaks are unusually good; this has shown up throughout the testing.

From these results it is evident that the claimed range of 50 km is conservative, and allows ample margin for poor weather and very bad lines.

Explanation of the I.F. sensitivity curve

Figure 2 is purely a curve of voltage at the input stage against readings on the normal AGC meter incorporated in the instrument. It does not take into account conversion loss. This curve was plotted by using a signal generator with a 5 ohms output impedance coupled to the I.F. unit through its normal cable; this was done so that the generator operated as a voltage source and hence voltage at the input stage could be reliably known.

The technical specifications of the MRB201 are given in Appendix 2.

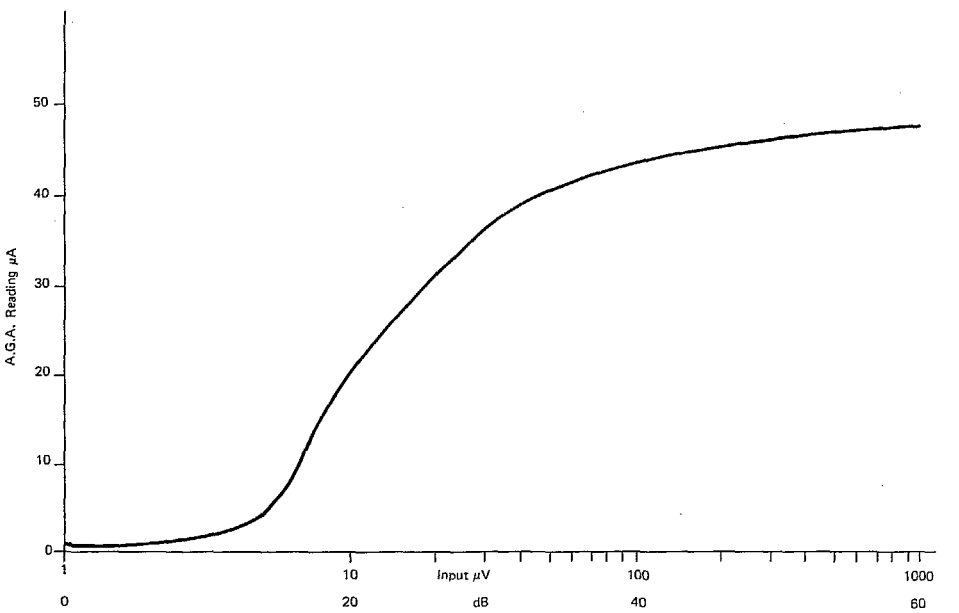


Figure 2 – *MRB 201 I.F. sensitivity. Unit No. 1355*

APPENDIX 1

Test No. 1: 23 June 1969, a.m.

Instruments No. 1355, in east tower
No. 1354, in laboratory tower

Range Factory test line length 105.597 m

Weather *Barometer* 772.2 mm
Humidity 70 %,
corresponding to partial pressure of 9 mm
Cloud 1/10
Wind slight SE

Note. The instruments were in early prototype form and the antennae were mounted on spacers hence a correction of + 10 cm had to be made for *each* instrument i.e. total + 20 cm. At this range meterological correction is insignficant.

Readings

		No. 1354	*No. 1355*
Cavity 596	A Forward	105	108
	Reverse	105	108
		2/105	2/108
	A Reading	052½	054
	D	104	104
	C	010	010
	B	000	000
		00105.2½ m	00105.4 m
	+ 20 cm correction =	105.4	= 105.6

Cavity 820 00105.5
+ correction = 105.7
Cavity 632 00105.0
+ correction = 105.2

AGC was above 45 μA on both instruments

Test No. 2: 23 June 1969, a.m.

Instruments as for test No. 1
No. 1354, on Plessey roof
No. 1355, approx. 8. km away

Range Plessey factory roof to arbitrary peg driven into parking area at top of nearby scenic drive – "Die Ou Kaapse Weg" – length not accurately known, > 8 km

Weather As for Test No. 1

Readings

	No. 1354	*No. 1355*
A Forward	124	123
Reverse	122	121
	2/1123	2/1122
	561½	561
D	058	056
C	800	806
B	078	078
	08056.1½ m	08056.1 m

AGC 45 μA

Test No. 3: 23 June 1969, p.m.

Instruments No. 1355, in laboratory tower
No. 1354, at Sir Lowry Pass

Range Factory long range test line length 44757.69 m

Weather *Barometer* 772.0 mm
Humidity 70 %
Cloud 2/10
Wind SE

Calculated refractive index for these conditions is N = 333
i.e. R.I. = 1.00033 3. Pattern frequencies have been calculated using a R.I. of 1.000330, therefore correction is -3×10^{-6}
$= 44757.7 \times -3 \times 10^{-6}$
$= 0.134$ m.

Readings		*No. 1355*	*No. 1354*
Cavity 500	A Forward	142	139
	Reverse	138	139
		2/1140	2/1139
		570	569½
	D	757	758
	C	477	478
	B	446	446
		44757.0 m	44756.9 m
Corrections	Met	− 0.13	− 0.13
	Instrument (2 × 10)	+ 0.20	+ 0.20
		44757.1 m	44757.0 m
AGC		40 + μA	40 + μA

Test No. 4: 21 July 1969

This was a test carried out to determine any cyclic error which might be present. For this test the line from the factory roof to the parking area on "Die Ou Kaapse Weg" was used, a point was selected on the very edge of the parking area to allow the instrument to be stepped back in increments of 10 m. Only five steps could be accommodated. The first step was measured as 11 m. in error. These are purely relative readings and hence no corrections are applied.

Instruments No. 1354, ou Kaapse Weg
No. 1355, on Plessey roof

Readings Note. No detailed readings are shown, only the results

Increment (metres)	*No. 1355* Readings	*No. 1355* Deviation from increment	*No. 1354* Reading	*No. 1354* Deviation from increment
0	8044.1	–	8044.1	–
+ 11	8055.4	+ 0.3	8055.5	+ 0.4
+ 21	8065.4	+ 0.3	8065.3	+ 0.2
+ 31	8075.5	+ 0.4	8075.2	+ 0.1
+ 41	8085.4	+ 0.3	8085.3	+ 0.2
+ 51	8095.4	+ 0.3	8095.4	+ 0.3

Test No. 5: 13 November 1969

This was a long range test carried out not for accuracy, but to determine general performance under low signal conditons.

Instruments No. 1355 laboratory tower
No. 1354 Du Toits Kloof pass

Range From laboratory to the top of Du Toits Kloof pass
Distance 66 + km

Note. the instruments were in their final form and no antenna correction was necessary

Readings		*No. 1355*	*No. 1354*
	A	2/ 811	2/ 798
		406	399
	D	342	338
	C	638	626
	B	668	652
		66340.6 Metre	66339.9 Metre
AGC		25 μA	17 μA

APPENDIX 2

TECHNICAL SPECIFICATION MRB201

Range 1 to 50 km assuming adequate line of sight conditions.

Accuracy ± 1 m under dynamic conditions. ± 0.5 m
± 3 p.p.m. of distance when used as static instrument with full set readings taken across frequency band and meteorological correction applied.

Resolution 0.1 m maximum resolution being usable only on land or under exceptionally quiet conditions.

Beamwidth As for existing MRB2

Master: Horizontal 27° Remote: Horizontal 24°
Vertical 22° Vertical 20°

Operators 2 to 3 depending on installation and system.

Readout

Fine: 6 digits (Nixie tubes)–giving integrated fine readings.

Digit reading 0.1 m can be covered during dynamic readings.

Coarse: Single centre zero meter which reads

Integrated digit being checked–
(Fine reading–coarse reading)

e.g. 100s digit–(A–D)

This meter reads for each coarse pattern in turn.

This meter to be calibrated – 2, – 1, 0, + 1, + 2 and to be suitably damped.
Each Nixie digit to have a suitable identification (probably lamp) which indicates which coarse digit the meter is coupled to.
Also, each digit to have a suitable switch (or button) to add or subtract one digit at a time.
Patterns to be A unambiguous to 100 m
A–D to 1000 m
A–C to 10 km
A–B to 100 km

Readout to have auxiliary output which gives standard binary coded decimal output of actual readings (not difference) to enable a tape punch, computer, plotter etc. to be driven.
Readout characteristics to be such that fine readings can be given either fast or slow slewing rate (7 sec or 70 sec) selected by switch on front panel. When on slow, readout will then be effectively smoothed; fast mode overrides in case of sudden change of speed or direction.

Measuring pattern frequencies

Master function:	A 14 98 470 Hz	Remote function:	A– 14 99 470 Hz
	B 14 96 972 Hz		A+ 14 97 470 Hz
	C 14 83 485 Hz		B 14 95 972 Hz
	D 13 48 623 Hz		C 14 82 485 Hz
			D 13 47 623 Hz

Note. These frequencies are calculated to give direct metre readout assuming an average refractive index of 1.000330.

Antenna All existing MRA2 and MRB2 antennae can be used normally. Master will use the rotating twin antenna and the remote will use the directly attached reflector and dipole.

Carrier frequency 2800–3200 MHz

Note. This may be moved to 3000–3400 MHz in future.

Carrier output power 100 mW

Current consumption 1 Unit 2–2$\frac{1}{2}$ amps at 12 volts

Readout 1 amp at 12 volts.

Configuration One or two channel as needed.

Size and weight Smaller than 1 ft cube and less than 30 lb.

Unit function Each unit to have dual master remote function, although to save cost units used as remotes on shore need not have the digital readout plugged in – a standard MRA3 dial readout can be substituted for baseline measurements.

Method of pattern selection

Master function: Switch on front panel *or* voltage control from external control unit.
Remote function: Tone gates opened by tones in range 40 kHz to 70 kHz radiated by master unit.

Speed of operation To be capable of being operated easily at up to 30 knots (15.45 m per/sec) within rated accuracy.

Note. To take readings within allowable error at full speed automatic reading equipment (e.g. type punch) will have to be used.
At maximum speed accuracy will degrade during rapid changes of direction.

Environmental conditions Operable within a temperature range of – 10°C to + 50°C. Lower limit is set by rotating antenna.

DISCUSSION

Strilaeff: You mentioned an accuracy of ± 1 m. Can this accuracy be improved at shorter distances?

d'Arcy: An accuracy better than ± 1 m is exceptional. With increasing distances out to sea, a larger area of errors surrounds the ship. A shorter range improves the accuracy.

Modern position fixing methods

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Abstract. Modern electronic position fixing methods are explained and the prerequisite conditions for automatic recording and evaluation are shown. Various forms of registration and the accuracy of position data are discussed. Propagation velocity is determined and comments are submitted on several methods. The paper concludes with a description of results obtained and experience gained with systems tested by the Federal Institute of Hydrology.

SYSTEMES MODERNES DE REPERES FIXES POUR ETUDES HYDROGRAPHIQUES ET HYDROLOGIQUES

Résumé. L'auteur expose divers procédés électroniques modernes de radionavigation et indique les conditions nécessaires à un enregistrement et une exploitation automatiques des données. Il soumet à un examen critique les différentes formes d'enregistrement et la précision de la radionavigation. Il détermine la vitesse de propagation et formule des remarques sur les différents systèmes. En conclusion il fait part des résultats obtenus et de l'expérience acquise avec des systèmes expérimentés par l'Institut fédéral d'hydrologie.

MÉTODOS MODERNOS DE POSICIÓN FIJA

Resumen. Se explican los métodos electrónicos modernos de posición fija y las condiciones previas para el registro y la evaluación automáticos. Se discuten diversas formas de registro y la precisión de los datos de posición. Se determina la velocidad de propagación y se comentan diversos métodos. La comunicación concluye con una descripción de los resultados logrados y de la experiencia obtenida con los sistemas ensayados en el Instituto Federal de Hidrología.

СОВРЕМЕННЫЕ СИСТЕМЫ ДЛЯ ФИКСАЦИИ ПОЛОЖЕНИЯ (ДЛЯ ГИДРОГРАФИЧЕСКИХ И ГИДРОЛОГИЧЕСКИХ СЪЕМОК)

Резюме. Описываются современные методы фиксации положения, основанные на применении электроники, и заранее определены требования для автоматической регитрации и определения величины. Рассмотрены различные формы регистрации и точность данных о положении. Определена скорость распространения и сделаны комментарии по нескольким методам. В статье содержатся описания полученных результатов и опыта эксплуатации, имеющихся в Федеральном институте гидрологии.

The systems for hydrographic surveys used by many authorities up to the present time are no longer sufficient for ensuring the security of traffic. A precise knowledge of the morphology of the sea bottom or the river bed is required. It is necessary both to increase the number of soundings and the precision of their positions and to evaluate the survey results faster. The aim must be to record survey data in a form that will allow the immediate calculation of the data by electronic computers and plotters. Close co-operation between users and manufacturers has resulted in the development of recording and evaluation systems so that this ideal is now a reality.

One of the basic prerequisites is the use of modern position fixing procedures, often called "precision position fixing" or "electronic survey systems". Longwave systems being used in ship's navigation are not sufficient for precision surveys because of their irregularities and propagation instability. Those systems use hectometre, metre, and decimetre waves. The systems can be classified in different ways, according to: (a) the main range of application; (b) the working frequency; and (c) the method of application.

The latter seems to be the most practical approach. Thus, one may distinguish between survey systems using circular base lines and those using hyperbolic base lines; the first-mentioned group can be subdivided further according to whether polar co-ordinates or bilateral systems are employed.

If users are asked about the position fixing precision they require, their answers are rarely exact. This is found especially in the sea waterways close to the coast, and in the estuary area. The small scales still used in drawing the bathymetric maps of these areas encourage a certain lack of precision. Deepening and maintaining of the waterways will lead, in some areas, to larger bathymetric map scales since this will allow the work to be executed more efficiently and at less expense. Thus it is appropriate to reflect on the influence of the "geometry" between the transmitter positions and the object which is to be fixed.

For position fixing by means of a bilateral system, distance-measuring instruments are used. The polar system also requires grid bearings. The precision of electronic distance measurements in sea surveys can be estimated at ± 1 m, the precision of the grid bearing depends on the sighting of the target (a lamp on the survey vessel), the estimate being ± 3°. If the required limiting value of ± 6 m in position fixing on rivers, channels, etc. is not exceeded, the mean error of a single fixing must be smaller than ± 2 m.

The polar method [1]

The allowable maximum distance (S_{max}; Fig. 1) is to be determined.

$$f_s \leqslant 2 \text{ m} ; f_x = \pm 1 \text{ m} ; f_\alpha = \pm 3^c$$

$$f_y = \sqrt{f_s^2 - f_x^2} = \pm 1.7 \text{ m}$$

$$S_{max} = \frac{\rho^c \cdot f_y}{f_\alpha^c} = \frac{6{,}366 \cdot 1.7}{3} = 3{,}600 \text{ m}.$$

The polar method can be used around the station A within the minimum distance S_{min} given by the electronic properties of the instrument, and the maximum distance S_{max} = 3,600 m. For optical following this might be the possible limit.

The bilateral method

The allowable intersection angle of the secants between the distance meters and the position to be fixed is to be determined (Fig. 2). $f_s \leqslant 2$ m; $f_x = \pm 1$ m.

It is well known from geodesy (the problem of intersection using determination elements containing errors)

continued on page 470

(1) Superscript *c* expresses geodetic angles in centigrades.

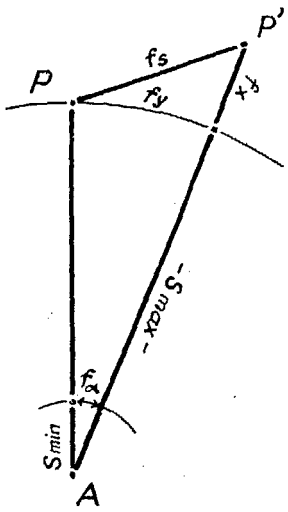


Figure 1 – *Polar method*

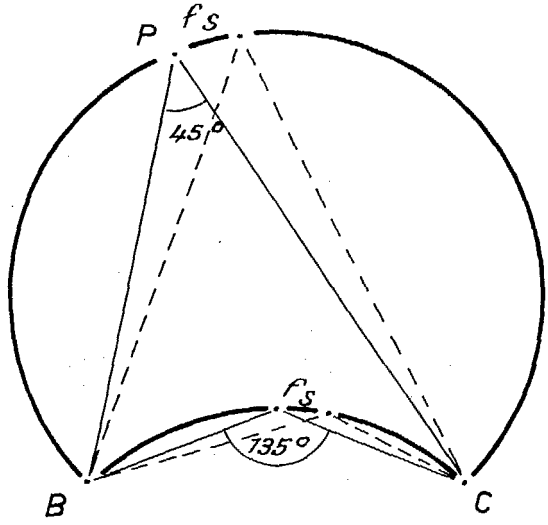


Figure 2 – *Bilateral method*

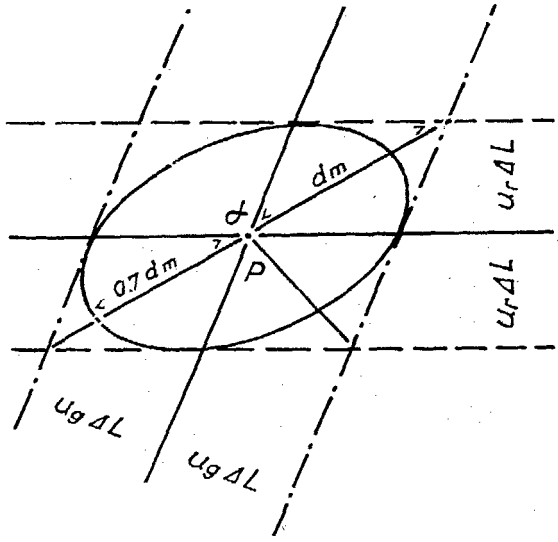


Figure 3 – *Hyperbolic method*

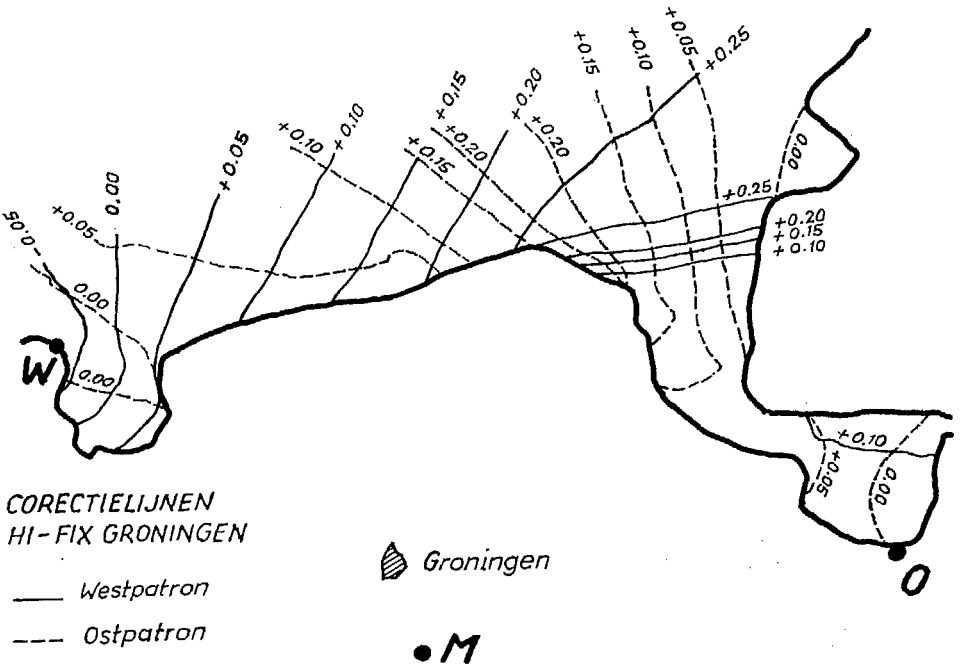


Figure 4 – *Correction lines for the North Sea, north-eastern part of the Netherlands*

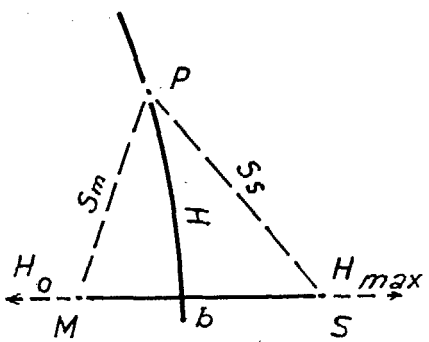


Figure 5 – *Determination of the propagation velocity*

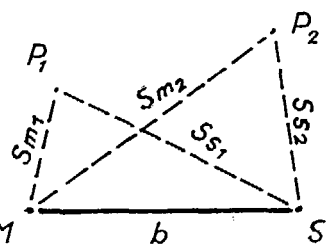


Figure 6 – *Determination of the propagation velocity*

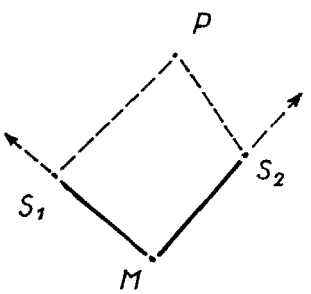


Figure 7 – *Determination of the propagation velocity*

$$f_s = \frac{1}{\sin \alpha} \cdot \sqrt{f_x^2 + f_y^2}$$

the errors being constant. Thus

$$\sin \alpha = \frac{1}{f_s} \cdot \sqrt{f_x^2 + f_y^2}$$

This leads to:

$$\sin \alpha = \frac{\sqrt{2}}{2} = \pm 0.7$$

$$\alpha_1 = 45^\circ$$

$$\alpha_2 = 135^\circ$$

The enclosed area for $f_S \leqslant 2$ m is limited to the area between the circles enclosing the connexion line between stations A and B as a secant, and the inscribed angles of which are 45° and 135° respectively. This demand leads to frequent station changes and an irrational area of double coverage when the ship's course is close to the coast and, therefore, close to the stations. The critical angles are often not respected, either too large or too small angles being used.

The hyperbolic method

The survey systems employed so far, with one exception, use medium to high frequencies (2 MHz). They are based on the principle of phase difference measuring. Zero hyperbolae representing the geometric base lines of all positions with phase differences $n \cdot 2\pi$ are marked on maps. If predictions are to be made about the expected precision of position fixing, one must take into account the fact that the distances between the hyperbolae (lanes) are not constant and that a "basic value" of the position fixing precision has to be determined first. For our own investigations, a distinction is usually made between relative position fixing (possibility of repetition) and absolute position fixing. Thorough tests in the North Sea and the Baltic Sea have proved the standard values $\Delta l = \pm$ 0.035 lane in relative and $\Delta l = \pm$ 0.068 lane in absolute position fixing respectively.

These standard values are only valid for transmitters located close to the coast and for positions in the sea area. Moreover, the section angle of the hyperbolae tangents at the fixed position is important in giving predictions. It depends on the "geometry" of the transmitters, and on the location of the fixed point within the system.

To permit the assessment of the probable limits, the ellipse of errors, well known in geodesy, has been inscribed on a parallelogram. The connexions can be seen from Figure 3. The direction of the axes is given by the section angle of the tangents, and the distance by the lane width together with the standard value of the deviation. As the ellipse of error in a plan is not a suitable means of representation, the distance between the crossing of the axes and the intersection of the long diameter *dm* with the ellipse has been introduced as the limit of position accuracy. The fact that the distance is 0.7 dm can be proved. Some simplifications for drawing are possible. The limiting value lines of position fixing accuracy thus obtained have been proved correct.

In the case of the transmitters being installed further inland, or if there is land between the transmitters and the point to be fixed, it will be advisable to increase the value of ΔL so as to avoid unexpected errors.

Before concluding the discussion of errors, I should like to mention an investigation carried out by van de Schaaf of the Rijkswaterstaat at Delft. The precision of a position fixing chain in the Netherlands had to be estimated for a hyperbolic system, the master transmitter of which is located far off the coast; communication between master and slaves included large sections crossing land (Fig. 4). Van de Schaaf assumes the propagation velocity on land to be approximately 299,300 km/sec, and on sea approximately 299,700 km/sec and he calculates "transit time differences" for the ray paths between assumed position points for the master on one hand and one slave on the other. Thus, values of probable uncertainties are obtained for the drawing of lines of expected limits. This is of great importance, since the above-mentioned value of $\Delta L = \pm\ 0.035$ lane cannot be used here.

For all modern position fixing systems (survey systems) the propagation velocity c is used as a scale. Together with the frequency used, it serves to determine the wavelength.

With all electronic range-finding instruments operating in the frequency band between 3 and 10 GHz the distance is determined by measuring the transit time of an electromagnetic wave between the ends of a line S which is marked by the instrument centres:

$$S = \frac{c_0}{n} \cdot \frac{T}{2}$$

where c_0 is the propagation velocity in a vacuum (c_0 = 299,792.5 km/sec), T the double transit time, and n the dielectric constant depending on the atmospheric conditions.

Range-finders in sea survey work are equipped with a direct-reading instrument based on a defined n. Often, $n = 1.00033$ is used; this value is rather high for operating areas in the North Sea and the Baltic Sea. Under extreme weather conditions, this may cause an error of ± 0.3 m at 20 km distance. In special cases, the correction can be effected by changing the modulation frequencies by 10-15 Hz.

The determination of the propagation velocity in hyperbolic systems using frequencies of about 2 MHz is more problematic.

If f is the frequency used, λ is the wavelength, a hyperbola number, S_m the distance between the master M and point P, S_s the distance between the slave S and P, b the length of the base line, i.e. the distance between M and S (Fig. 5). Now

$$\lambda = \frac{c}{f}\ ; \qquad H = \frac{1}{\lambda} \cdot (S_m - S_s + b).$$

This shows that the zero point for counting the hyperbolae is located in the master transmitter M (the extension of the base line beyond the master station M is called the hyperbola $H = 0$). It is also obvious that the greatest hyperbola number H_{max} arises in the extension MS beyond S.

1. Determination of c. When the extension of the base line MS is exceeded beyond M, the counter of the survey system receiver will show the minimum reading; when the other extension is exceeded, the maximum will be shown; thus H_{max} is known.

It follows from

$$H_{\max} = \frac{b}{\lambda/2} = \frac{2b}{\lambda} = \frac{2 \,.\, b \,.\, f}{c}$$

that

$$c = \frac{2 \,.\, b \,.\, f}{H_{\max}}.$$

During a test carried out in the German Bay in 1964-65, c was determined in this way for one pair of transmitters as $c_r = 299{,}568$ km/sec and for the second pair as $c_g = 299{,}664$ km/sec. The difference in the propagation velocities is surprising; moreover, both values are lower than expected on the grounds of previously available information. This method gives only one value (also in the case of repetition). It is not possible to determine by this method the error of the zero point (probably the electronic phase error of the slave or reference transmitter). This error is contained in $H_{\max}$ and falsifies the result. The method cannot even be recommended for the determination of approximate values. If readings are taken at distances shorter than 5λ, they must be corrected.

2. *Synoptic measuring* at several points is another method of determination.

H_{a_i} = readout hyperbola number;
H_{w_i} = true hyperbola number;
n = zero point (phasepoint) error of the slave.

From the Figure 6 it follows that

$$H_{a_1} = H_{w_1} + n = \frac{1}{\lambda}\,(S_{m_1} - S_{s_1} + b) + n,$$

$$H_{a_2} = H_{w_2} + n = \frac{1}{\lambda}\,(S_{m_2} - S_{s_2} + b) + n,$$

$$\Delta H = H_{a_1} - H_{a_2} = H_{w_1} - H_{w_2} = \frac{1}{\lambda} \,.\, [(S_{m_1} - S_{s_1}) - (S_{m_2} - S_{s_2})].$$

It follows that

$$\Delta H = \frac{1}{\lambda}\,(\Delta S_1 - \Delta S_2) = \frac{f}{c}\,(\Delta S_1 - \Delta S_2)$$

$$c = f \cdot \frac{\Delta S_1 - \Delta S_2}{\Delta H}.$$

The result of a series of observations of the same chain mentioned above is

$c_r = 299{,}601$ km/sec (previously: 299,568 km/sec);
$c_g = 299{,}797$ km/sec (previously: 299,664 km/sec).

As the error of the zero point is without any effect in this calculation, other causes might influence the results. Probably, the accuracy in determining H does not correspond with the increasing factor given by

$$\frac{\Delta S_1 - \Delta S_2}{\Delta H}.$$

3. A third possibility consists of taking readings at a co-ordinated point and determining c from the differences resulting from a comparison of these readings with those obtained in the extension of the base lines (Fig. 7). Here, too, the results are most improbable.

Theoretically, the reading in the extension of the base lines must be constant, but this is not the case in actual practice. The results may be improved by taking the readings synoptically.

4. A fourth method has been found to give satisfactory results. It provides for a number of points to be established geodetically and corresponding hyperbolae numbers to be determined which should be scattered over a longer time interval.

It is assumed that

$$H_i + n = \frac{P_i \,.\, f}{c_0 + d_c}$$

where H_i = hyperbolae number, $P_i = S_m - S_s + \mathrm{b}$, the effective distance, f = frequency, c_0 = approximate value of the propagation velocity, d_c = correction to approximate value c_0.

Taking $n \,.\, d_c \ldots \longrightarrow 0$ leads to: $n \,.\, c_0 + d_c \,.\, H = \mathrm{P} \,.\, \mathrm{f}$. From here n and d_c can be determined without difficulty. The estimate can also be adjusted. For the already mentioned chain, the following values were obtained by this method:

$c_r = 299{,}745$ km/sec, $\quad n_r = -0{,}18$ lane,
$c_g = 299{,}780$ km/sec, $\quad n_g = -0.125$ lane.

For these surveys, oil drilling platforms with well-known co-ordinates could be used as fixed points. "Runaways" were encountered at one lighthouse. The quoted values are mean values of one year and apply to the sea area. The hyperbolic systems employed in estuary regions led to the following evaluation results:

$c_r = 299{,}645$ km/sec, $\quad c_g = 299{,}620$ km/sec,
$c_r = 299{,}710$ km/sec, $\quad c_g = 299{,}680$ km/sec.

Previously, the propagation in shallow places was found to be less than on the open sea.

A last practical example is of a survey system in the frequency band of about 2 MHz which allows hyperbolic and circular base lines to be used. Ship steering controlled by this system using circular baselines has been demonstrated on a river (simultaneously an example of "electronic mooring" was shown).

The authority which had given the approval expected to gain a position fixing system for hydrographic and hydrological operations. For this purpose, several positions had been prepared, and hyperbola numbers had been calculated from the co-ordinates of the transmitters. The disappointing results showed "dislocations" between 104 m and 58 m. Using a frequency of 1.7 MHz a propagation velocity of 299,000 km/sec has been assumed.

An adjustment by means of method 4 showed that $c_r = 298{,}629$ km/sec; $n_r = +29.2$ m; $c_g = 298{,}699$ km/sec; $n_g = +48.9$ m.

The use of these data lead to "dislocations" between 1.1 m and 11.0 m. This example especially may demonstrate the importance of the "scale" (i.e. the propaga-

tion velocity corresponding with the production of a map), and the determination of the "zero point error".

The differences (after adjustment) might have been reduced to half by investigating the "repeatability" of "absolute position fixing". The position precision work on rivers is different in every case, and has to be determined. The use of the systems mentioned allows the recording of "areas"; at the moment many authorities are still working on "profiles".

The operation, including the use of instruments with circular base lines, is controlled aboard the survey vessel. Where hyperbolic base lines are used, the operation of a monitor station is recommended. It can be installed so that the system may be controlled without additional personnel. As the monitor indicates the breakdown of a transmitter, the damage can be quickly repaired. The monitor recordings show when atmospheric disturbances lead to mislocations so that the measurements must be interrupted or corrections recommended.

Before discussing the systems, there are still some remarks to be made. Systems using circular base lines operate—with one exception—on frequencies of about 2 MHz.

Up to now they have allowed only one ship's position to be fixed. The automation of the land stations is not quite complete, and the aerial directions of these stations must be corrected if the ship moves outside the fixed area. The position of the land stations has to be changed often. The precision of the surveys is very high.

Systems using hyperbolic base lines are able to fix the positions of an unlimited number of ships. The stationary transmitters work without any interruption, and they need little servicing. The accuracy is less than that of circular systems.

In addition to the position fixing precision, depending on the task to be executed, it is of great importance for the users whether there is a good service in the country as well as a sufficient area of "good base line intersections". Using hyperbolic base lines the installation conditions demand either free or bound hyperbola chains. Free chains must have more equipment. Survey systems working on medium to high frequencies require quasi-optical sight conditions in the meter-range.

In many countries the decision on the system to be used depends on the number of required frequencies, their bandwidth, and the minimum output, especially in the 2 MHz band. It is often necessary to get special licences for this purpose.

The names of a number of survey systems used in hydrological and hydrographic work are well known. Thus a brief description of their characteristics will suffice.

Ralog is an instrument using the method of polar co-ordinates with a zero grid bearing. The fact that deviations from the zero-direction cannot be registered is not satisfactory. The shore to distance recorders of Atlas and Fahrentholz, too, must be mentioned here. They are used in position fixing when employing bottom profile recorders.

For position fixing by polar co-ordinates, no systems are known so far, but some developements have been announced. The problem is not the distance measurement, but the angle measurement. Following the target by a theodolite, which is able to measure light electrically and to transmit its values directly aboard, is very exacting for the observer, leading to fatigue and decreasing precision. The method will be much better if it is possible to pick up bearing changes electronically, and the precision will satisfy the requirements. This means working under weather conditions (bad sight conditions) which at the moment allow distance measurement, but not the measurement of angles.

The *Hydrodist* of Tellurometer (U.K.) Ltd., and the *Autotape* of Cubic are available for use with circular base lines. Both are in use world-wide. Experience with

the Hydrodist has been collected in the coastal area for the last 10 years. The system requires a large staff. Repeatedly the opinion has been voiced that without these instruments it would be difficult, if not impossible, to perform several tasks. The firm has worked for some time on a perfected system, which will be demonstrated at this symposium. This system will allow the connexion of peripheral instruments. It is of great importance that counting need no longer be started again in case of sight breakdown between land station and ship; the "starting distance", too, is now determined automatically.

The Autotape allows the same continuous counting and automatic determination of the "starting distance". Peripheral instruments can also be connected with this equipment and the same operations can be executed as with the Hydrodist. It may be mentioned that trials have been performed to fix the positions of many ships. Its use can be recommended if a sea area and a long base line will allow working in a large area of good intersections. Due to the operating frequency 3 GHz) used, both systems require quasi-optical sight. Thus the range depends not so much on electronic (power) and atmospheric circumstances but, rather, on the antenna altitude of the instrument stations.

For hyperbolic base lines several systems are available: HiFix with different transmitter types, Lorac, Rana, including the special type Ragep, Raydish, and Toran. They work—with one exception—on frequencies of about 2 MHz. The following data may be of any interest to the user.

HiFix requires one frequency with a band width of + 60 Hz, the normal output being 10 watts when used in the offshore area. The system has been thoroughly tested here. Several chains are operating in the Federal Republic of Germany, one is under construction. The crews of the surveying vessels like the system. In some districts complementary equipment has been added.

Lorac requires two frequencies with a given minimum spacing, and a large bandwidth. The requisite output, too, is larger than that of comparable systems. Lorac is an excellent navigation and survey system in areas far off the coast.

Rana also required two frequencies and a wide bandwidth. It was demonstrated once in the Eckernförde Bay with satisfactory results. The equipment was bulky and the requirements for installation in densely populated areas could be met rarely. The hyperbola chains are free (see Ragep and Toran). Depending on the possibility of moving the stations on the land, this leads often to a larger working area of good line intersections. As the instruments require little service, the more bulky equipment is significant.

Ragep is a development of Rana. This system operates on medium waves. As the lanes are narrow, lane identification is of special interest and for this purpose special frequencies are required. This means 5 frequencies (total) in a band reserved in some countries (e.g. Federal Republic of Germany) for special security services. Bulky equipment with a critical output makes it major task offshore position fixing. Reduced in sizes, Ragep could be an interesting survey system for rivers and channels.

Raydist is almost unknown in Europe and was demonstrated in a miniaturized form probably only once, during a hydrographic conference in Monaco. It consists of small, handy instruments which can be easily transported. Two frequencies with small bandwidth, and a low output, are required. The possibilities of the system are manifold. Of greatest interest is the possibility of using one hyperbolic and one circular base line simultaneously.

Toran requires two frequencies with a difference of 10-15 kHz and bandwidth of 80 Hz and 200 Hz. The output can be regulated due to the local conditions. Trials

have been recently conducted on the Unterelbe, the frequencies used being about 1.65 MHz and 1.97 MHz. During reconnaissance work and installation of land stations, the possibility of setting up the hyperbola chains independently was proved again.

The introductory remarks showed modern position fixing systems to be a prerequisite of automatic drawing of maps and bathymetric plans. Several authorities could already execute automatic river profile recording as well as automatic drawing of "point maps" including depth data. Automatic drawing of bathymetric lines is a different problem, but there is already the possibility of doing this, as will be shown in another paper in this symposium.

The above-mentioned remarks should show that position fixing systems have been developed by industries which allow an increasing number of points to be fixed more precisely for getting better hydrographic and hydrological data. It is necessary to know fully the limits of the system, and to fulfil some prerequisites for making the best use of the precision systems.

The precision of electronic survey systems is dependent upon internal and external errors. The internal error depends on instruments precision being similar in all systems, the external error depends on geometry and the characteristics of propagation velocity.

DISCUSSION

Laurent: Pourquoi ne peut-on éliminer l'erreur du "point zéro" en effectuant la mesure de la phase minimale (sur le prolongement de la base au-delà d'un foyer), et de la phase maximale (sur l'autre prolongement de la base) ? On a alors

$$c = \frac{2 \,.\, b \,.\, f}{H_{\max} - H_{\min}}$$

$H_{\max}$ et $H_{\min}$ peuvent être déterminés à ± 1/100 pratiquement.

Lohrberg: I regret that I am a geodetic surveyer and not an expert on electronics. The method you mention is unknown to me, but I believe that it is extremely important to know the velocity as well as the error of the "point zero". This error does not influence the velocity, but is essential for obtaining correct results.

Rodier: Pour déterminer la position du point dans des sections de 300 m à 3 km de largeur nous utilisons le cercle hydrographique bien connu des hydrographes (on l'appelle également alidade). Cet appareil, très simple et robuste, est suffisamment précis pour des cours d'eau tels que l'Oubangui ou le Congo. Il nécessite qu'on pose des balises sur les deux rives pour effectuer les visées.

The application of an orientation system for continuous observations for the compilation of river channel maps by means of an echo-sounder

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Abstract. This paper presents a method for the determination of the horizontal position of sounding points by the use of two angles of resection. The angles are estimated by the continuous observation of permanent shoremarks by means of three layout devices installed on the deck of the sounding vessel. Angles of resection are obtained by using a system comprising self-synchronizing sensors and differential self-synchronizers. The horizontal position of sounding points is fixed on the map on board the moving ship. The depths measured by the echo-sounder are recorded simultaneously. The time-lap between field and office work is eliminated.

APPLICATION D'UN SYSTEME D'ORIENTATION A OBSERVATIONS CONTINUES AU LEVE TOPOGRAPHIQUE DU LIT FLUVIAL, AU MOYEN D'UN SONDEUR A ECHO

Résumé. Cet article présente un procédé pour déterminer la position des points de sondage sur une rivière à l'aide de deux arcs capables. Ces arcs capables sont déterminés par l'observation continue de points de repère fixes placés sur la rive à l'aide de trois installations de visée établies sur le point du bateau-sondeur. Les arcs capables sont obtenus par l'application du système : selsyns-transmetteurs-selsyns différentiels.

La position des points de sondage est déterminée sur le plan pendant la marche du bateau. Simultanément on enregistre les profondeurs mesurées par le sondeur à écho. L'intervalle de temps séparant les travaux sur le terrain (levé) et les travaux de bureau est réduit à zéro.

APLICACIÓN DE UN SISTEMA DE OBSERVACIONES CONTINUAS PARA LA COMPILACIÓN DE MAPAS DE CAUCES FLUVIALES POR MEDIO DE ECOSONDAS

Resumen. La presente comunicación describe un método para la determinación de la posición horizontal de los puntos de sondeo utilizando dos ángulos de marcación. Los ángulos se estiman por la observación de marcas permanentes en la orilla por medio de tres dispositivos de alidadas instalados en la cubierta de la embarcación de sondeo. Los ángulos de marcación se obtienen usando el sistema selsins-sensores-selsins diferenciales.

La posición horizontal de los puntos de sondeo se fija en el mapa directamente en la embarcación y simultáneamente se registran las profundidades medidas por la ecosonda, eliminando el intervalo de tiempo entre el trabajo en campo y en gabinete.

ПРИМЕНЕНИЕ СЛЕДЯЩЕЙ СИСТЕМЫ ОРИЕНТАЦИИ ДЛЯ СОСТВЛЕНИЯ ПЛАНА РУСЛА В ПРОЦЕССЕ ЕГО ПРОМЕРОВ ЭХОЛОТОМ

Резюме. Излагается способ определения планового положения промерных точек с использованием двух углов обратной засечки, получаемых при непрерывном слежении за постоянными береговыми ориентирами с помощью трех визирующих устройств, установленных на палубе промерного судна. Углы обратной засечки получаются благодаря использованию системы : сельсины — датчики — дифференциальные сельсины.

Плавоное положение промерных точек фиксируется на планшете в процессе движения судна. Одновременно выписываются глубины, измеренные эхолотом. Ликвидируется разрыв по времени между полевыми и камеральными работами.

Among the different means for fixing sounding points, methods based on geodesic resection have found extensive application. A sextant is mainly used as a goniometrical instrument. Points are plotted on the map by means of a protractor using special goniometrical networks. The points of the control network serve as landmarks.

In general, for resection from a moving vessel, it is necessary to determine two angles, α and β, at any given moment. The vertices of the angles α and β lie at the sounding point 0, and the sides of the plotted angles pass through the three landmarks on shore (I, II, III; Fig. 1).

The accuracy of the determination of the location and the number of sounding points depends on the frequency and synchrony of the measurement of the angles α and β. Direct measurement of angles from a moving vessel is possible only with the aid of two sextants. However, strictly synchronous sightings by two observers is difficult to achieve, and the necessity of recording the readings prolongs the interval between two consecutive readings.

The operation of geodesic instruments needs specially trained observers. Recently the method of fixing sounding points by automatic resection has become widespread for river surveys. The basis of the method is an observing system (Fig. 2) consisting of a layout instrument, a self-synchronizer (selfsyn; C) and a differential receiving selsyn (DC). When sighting the layout instruments on shore landmarks the rotors of the transmitting selsyns (C) begin to move; the shafts of the rotors are rigidly connected to the vertical axes of the display instruments.

The rotors of the transmitting selsyns are electrically coupled to the rotors of the differential receiving selsyns, which are the basis of secondary devices indicating the angles α and β.

The preliminary adjustment of the system consists of fixing the pointers of the two secondary devices on the zero graduation, while the sighting axes of the instruments are parallel.

Layout instruments are placed on an observation platform on board the sounding vessel with all-round visibility. Ideally, all three instruments should be placed at one point or on one vertical line. Stanchions on which the instruments are mounted are placed at the corners of a triangular frame. The length of the sides of the triangle depends upon the minimum distance necessary between the seats for the observers. The seats must turn around freely on the stanchions.

Secondary devices for indicating angles of resection are mounted on a control desk in the saloon of the vessel or in the wheelhouse. The insertion of sounding points and depths on the chart is undertaken by the operator supervising the whole survey.

The survey is carried out as follows. When the vessel is approaching the survey strip, the operator points out the landmarks to the observers and explains their order as the vessel moves along the strip. The feed circuit of the stator windings of the transmitting selsyns and the echo-sounder are switched on. The observers then point the display instruments at these landmarks and follow them continuously. At the same time, the secondary instruments on the control desk indicate the changes of angles of resection. Using the goniometrical network plotted on the chart beforehand, the operator fixes the approximate position of the vessel. When the vessel reaches the edge of the sounding strip, the feed circuit of the stator windings of the transmitting selsyns is switched off. After this the pointers of the secondary instruments stop and the operator reads the resection angles α and β. From these angles a point is marked on the chart which corresponds to the position of the

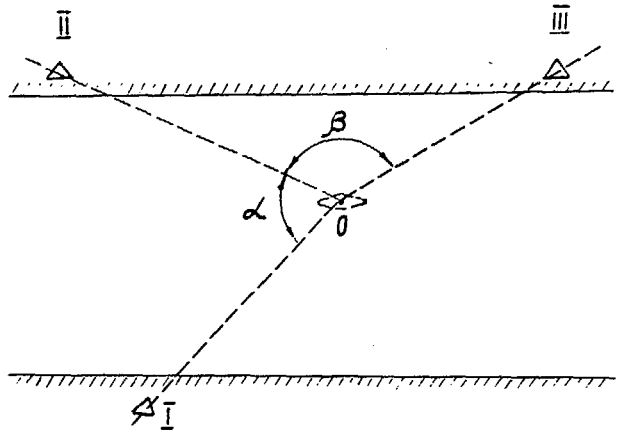


Figure 1 – *Scheme of geodesic resection*

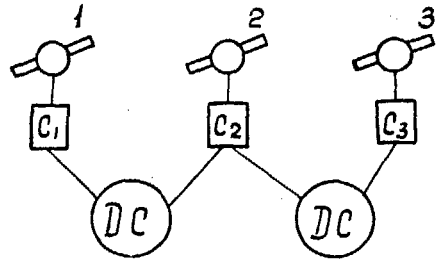


Figure 2 – *Block diagram of an observation system*

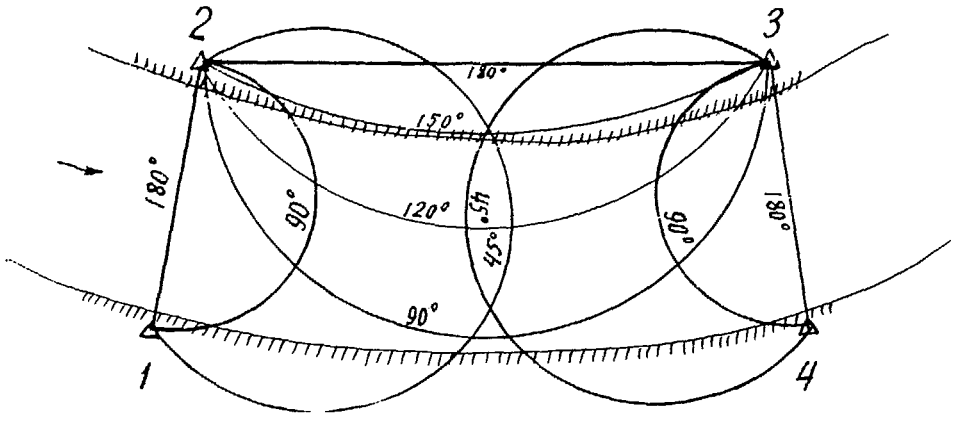


Figure 3 – *Example of distribution of landmarks and the goniometrical network*

sounding vessel at the given moment. On switching off the selsyn feed circuit, an operation mark appears on the bathygram of the echo-sounder. The values of depth are recorded on the chart at the corresponding points. For a certain time interval, which is determined by a time relay, the feed circuit of the selsyns is switched on. The pointers of secondary instruments begin to move and indicate new resection angles corresponding to the rotor position of the transmitting selsyns which turn with the vessel's movement, as the observers continue to follow the landmarks. On pressing a button on the control desk, all the operations are repeated.

When the end of the operation zone for one group of landmarks is reached, the operator, by a sound signal or by telephone, orders the observers on the observation platform to change landmarks and continues to plot the sounding points on the goniometrical network. The distance between the landmarks and their location is determined by the topographical characteristics of the sounding strip, by visibility and also by the requirements of the minimum possible resection angles. The minimum possible angle determining the accuracy of fix of the sounding points depends on the precision of the selsyns used.

When plotting a goniometrical network as a system of arcs with a common chord, the distance between the arcs, corresponding to the same increase in the angle, increases quickly as the angle decreases. Therefore, the use of angles of resection less than 30°-35° is not recommended. It is possible to avoid lesser resection angles, if the scheme shown in Figure 3 is used during the distribution of landmarks and the plotting of the goniometrical network. With this scheme, two transverse bases (1-2) and (3-4), will correspond to one longitudinal base (2-3). In surveying the left part of the strip points 1, 2 and 3 are sighted and, for the right part, points 2, 3 and 4 are sighted.

CONCLUSIONS

1. The method of automatic resection is based upon the use of an observation system consisting of a display instrument, a transmitting selsyn and a differential selsyn.
2. If natural or artificial landmarks are chosen in advance and fixed, there is no need for field operations on shore before or during sounding.
3. The combined use of an observation (following) system and an echo-sounder permits derivation of a river channel map during sounding.

IV Depth measuring
Mesure de la profondeur

Digital measurements of river bed profiles using a general-purpose data acquisition system

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Abstract. A computerized data acquisition system for measuring bed profiles is described. The system incorporates an IBM 1801 digital computer and Bludworth sounder which communicate, via a magnetic tape recorder and various other components, to provide punched cards with numbers representing the depth for each sounding. Interface hardware and programming techniques to establish synchronization between the computer and sounder are presented. Emphasis is not on describing the ultimate system, but rather on illustrating how the general-purpose data acquisition system can be adapted to a specific problem using available instruments.

DIGITALISATION DE LA MESURE DES PROFILS DU FOND D'UN FLEUVE PAR UN SYSTEME GENERAL DE COLLECTE DE DONNEES

Résumé. Cet article présente un système permettant d'obtenir certaines données à partir de la mesure des profils du fond d'un fleuve. On utilise à cette fin un ordinateur IBM 1801 auquel est reliée une sonde ultrasonique Bludworth par l'intermédiaire d'un enregistreur à bande magnétique et d'autres appareils complémentaires. On obtient ainsi des cartes perforées avec les valeurs numériques pour les différentes mesures de la profondeur. L'auteur décrit l'appareillage d'interface et les méthodes de programmation qui permettent la synchronisation entre l'instrument de sondage et l'ordinateur, en mettant l'accent non sur le système particulier dont l'auteur s'est servi, mais plutôt sur les aspects généraux du système d'obtention des données et sur la façon de l'adapter à un problème spécifique.

MEDICIONES DIGITALES DE PERFILES DE RÍO USANDO UN SISTEMA DE OBTENSIÓN DE DATOS PARA FINES GENERALES

Resumen. Se describe un sistema con ordenador para la medición de perfiles. El sistema incluye un ordenador digital IBM 1801 y un sondador Bludworth que se comunican por medio de un registrador de cinta magnética y otros diversos componentes, a fin de obtener tarjetas perforadas con números que representan la profundidad de cada sondeo. Se presentan las técnicas de hardware interfacial y de programación, a fin de obtener la sincronización entre ordenador y sondador. No se ha intentado describir el sistema final, sino más bien ilustrar cómo se puede adaptar el sistema para fines múltiples de adquisición de datos a un problema específico, utilizando los instrumentos disponibles.

ЦИФРОВЫЕ ИЗМЕРЕНИЯ ПРОФИЛЕЙ РЕЧНОГО ДНА С ПОМОЩЬЮ СИСТЕМЫ ДЛЯ ПОЛУЧЕНИЯ ОБЩИХ ДАННЫХ

Резюме. Описывается система получения цифровых данных при измерении профилей дна. Система включает цифровую вычислительную машину IBM 1801 и глубинометр (эхолот) Блюдворта, которые связаны посредством самописца с магнитной лентой и различными другими компонентами, обеспечивающими получение перфорированных карт с числами, характеризующими глубину дла каждого промера. Предложен способ установления синхронности между вычислением и зондированием. Упор делается не на описание законченной системы, а скорее на иллюстрацию того, как система для получения общих данных может быть приспособлена к частной проблеме, используя имеющиеся приборы.

INTRODUCTION

The problem of measuring bed profiles in alluvial channels has been approached many times with many different solutions, some having been simple and straightforward while others were more sophisticated. The system described in this paper is in the latter category, since it includes the combination of a stored-programme digital computer with sonic depth-measuring equipment. Briefly, the objective was to design an instrumentation system capable of measuring bed profiles with sufficient accuracy and resolution to permit meaningful computation by digital techniques of the statistical properties of bed forms. Thus, the instrumentation system for this study became a combination of instruments which individually were adequate only for one phase of the problem.

The basic instrument for measurement of bed profiles utilizes the principle of sound navigation and ranging (SONAR). A sound wave which is generated by the appropriate transducer and electronic circuitry, transmitted toward the channel bottom, reflected and detected, serves as the medium for determining time intervals proportional to the distance between the transducer and the channel bottom. These time intervals may be processed by several different procedures, depending on the information desired. If a continuous record or voltage is required, appropriate electronic techniques may be employed to convert the time intervals to voltage levels. The voltage, however, having been derived from samples, is really not continuous (although after effective filtering it may appear to be so) and contains no information not already contained in the sampled data. In fact, if the analyses to be performed are done in digital form, and thus require the data to be in digital form. the conversions from sampled data to continuous back to sampled data would only decrease the signal-to-noise ratio. Therefore, when the data are to be processed by digital techniques, it is advantageous to preserve them in sampled form. Also, by not having to convert the data to continuous form, more reliability and reduced weight in power requirements are realized for field instruments.

DISCUSSION OF LOGIC CIRCUITS FOR COMPUTING TIME INTERVALS PROPORTIONAL TO DEPTH

Figure 1 is a schematic diagram of the complete system with the IBM 1801 serving as system controller. Because the digital processing is not done in the field, it is necessary to record the output signals from the sounder on magnetic tape. The decision to process the time intervals directly required that an accurate oscillator with sufficient resolution be gated by the transmitted and echo signals; in order to avoid the effect of possible time variations introduced by the tape recorder, the ungated oscillator signal is recorded simultaneously with the time interval markers from the sounder.

The sequence of operation starts with the transmitted signal from the sounder which opens the AND gate controlling the oscillator output. The counter totalizes the pulses from the oscillator until the echo signal, which closes the AND gate, is received. At this point in time, the analogue-to-digital converter (ADC), via its external "Sync" capability, is instructed by the interface circuitry to convert the sixteen channels of the multiplexer containing the binary-coded decimal (BCD) information from the counter. The computer decodes the sixteen logic signals, stores the equivalent binary word for punching, resets the counter, and then returns control to the interface circuitry to wait for the next sounding or sample. After twelve

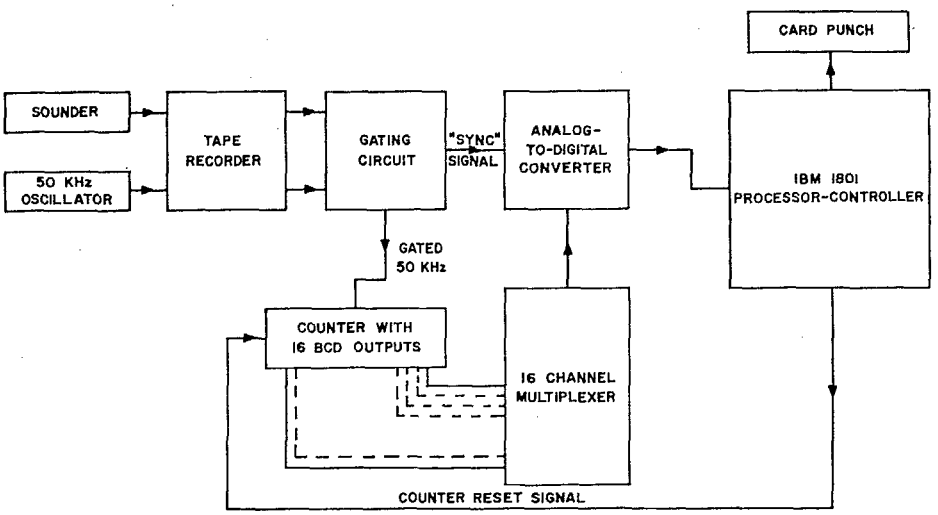


Figure 1 – *Block diagram of system for measuring bed profiles*

samples have been taken they are punched on an IBM card, with programme execution such that punching causes no loss of samples.

The sounder is a Bludworth depth recorder, Model ES-1025, and the time markers are obtained from the output of the receiver amplifier. To attenuate the high-voltage output from the sounder to an acceptable level for the tape recorder, a non-linear network (Glover, 1968) connects the output of the Bludworth to the recorder. The tape recorder used for collecting the field data is a 7-track Lockheed Model 417 equipped with direct record-reproduce electronics and one-voice edge track. Two data tracks record the sounding signal from the Bludworth and the 50 kHz oscillator signal, the latter being derived from a Model JB15-12Z crystal oscillator manufactured by Accutronics, Inc.

The problems associated with using the time markers from the ultrasonic equipment to gate the 50 kHz signal are many. However, because in general the depth of the water is 4 ft (1.2 m) or greater, certain freedoms are taken which for shallower depths would not be permitted. To aid in the understanding of the sequence of events for obtaining the data and of the reasons for including certain operations, reference will be made to Figure 2, which shows all pulses and time intervals of the various gates. The first and last traces are oscilloscope photographs of the recorded Bludworth signals and the gated oscillator signal which is fed to the electronic counter. Figure 3 is a block diagram of the interface circuitry; detailed circuit diagrams of the blocks in this figure are given in Glover (1968).

In general, the recorded signal is at the proper amplitude, but on occasion it needs additional amplification for correct detection. It is, therefore, amplified and then rectified prior to triggering the amplitude level detector or Schmitt trigger. The output from the Schmitt trigger sets the binary divider which in turn opens the AND gate to pass the 50 kHz signal. To eliminate the remaining pulses of the transmitted signal from influencing the binary divider, a 2 millisec pulse initiated by the divider is fed back to the OR gate controlling the Schmitt trigger. By eliminating the remaining pulses in this manner, the filter for smoothing the rectified transmitted signal is also eliminated.

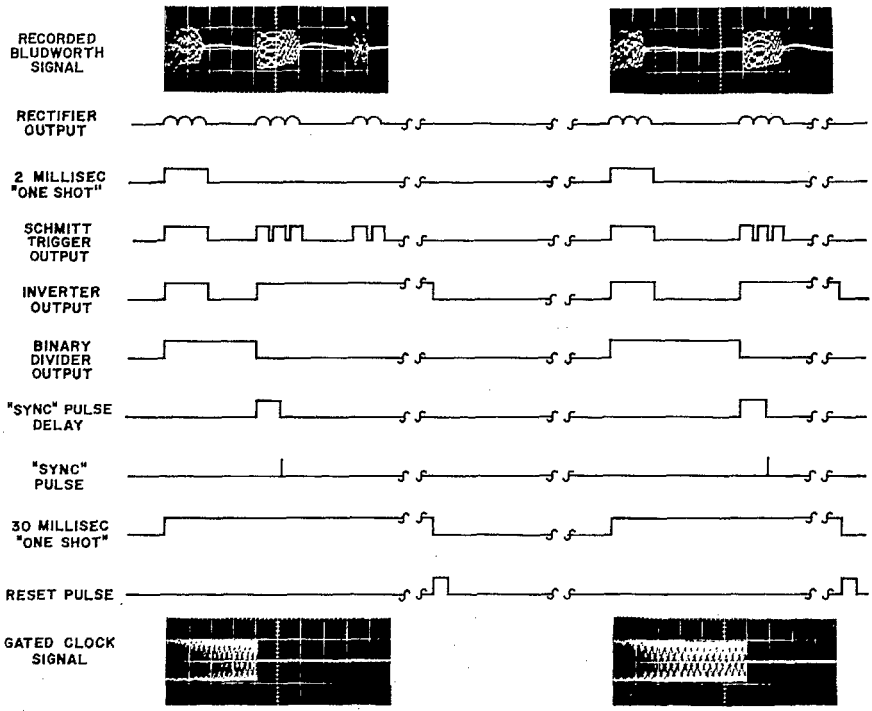


Figure 2 – *Response and time intervals of components in the gating circuit*

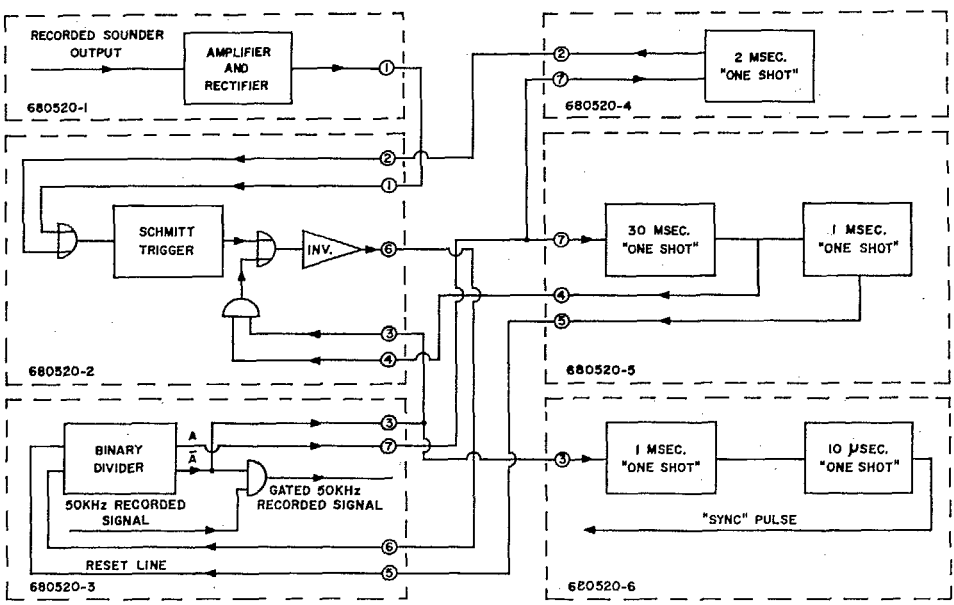


Figure 3 – *Block diagram of interface circuitry*

To remove the influence of the remaining pulses of the first and second echo signals, a 30 millisec pulse, initiated by the binary divider, and the complement of the divider, control an AND gate, the output of which is one of two signals controlling the inverter driving the binary divider. This controlling feedback exists only while both signals are logically true, because of the AND gate ahead of the OR gate feeding the inverter. Once the 30 millisec pulse terminates, the Schmitt trigger is again effective in setting the binary divider. In addition to performing this function, the end of the pulse initiates a 1 millisec pulse which resets the binary divider if an echo signal is not detected.

Synchronizing the computer so that it will read the BCD number from the counter at the proper time is accomplished by programming the ADC for external synchronization and by supplying the "Sync" pulse after the first echo is detected. To avoid transient conditions in the counter, a 1 millisec interval is inserted prior to generating the "Sync" pulse (10 microsec in duration) which starts the conversion.

DISCUSSION OF COMPUTER PROGRAMME FOR READING AND PUNCHING THE TIME INTERVALS

A flow chart of the Assembler computer programme for processing the magnetic tapes is shown in Figure 4. It does not include all details of the programme, but does represent its general operation and complete philosophy. A step-by-step explanation will not be given here, but several features will be discussed, because of their importance to the success of the programme.

The programme objectives are, first, to provide punched cards with data for twelve soundings per card and, second, to determine the number of soundings between fixes so corrections for uneven spacing between samples can be made. Since each pass through a reach gives about 6,000 soundings, storage of all points per pass prior to punching them on cards is not possible, the limitation being inadequate computer memory. Hence, a programme which can store the data from the tape recorder in lots of twelve and punch them on cards without interrupting the input process was developed.

The analogue-to-digital converter is programmed for external synchronization in order to provide the synchronization between ADC reading and sounder sampling. When operating in the external "Sync" mode, the ADC cannot convert a data word until the "Sync" pulse has been received, even though it has been instructed to do so by the computer. A test for completed ADC conversion follows the ADC convert instructions, and causes the programme to loop until the "Sync" pulse is received or synchronization is realized. Once the "Sync" pulse has been received, indicating that the first echo has been detected, the ADC converts the sixteen terminals. Sounding rates are slow enough and the remaining programme execution time is fast enough to give the instructions to convert the terminals well ahead of the point in time when needed.

The card punch, being a relatively slow output device, operates with the computer jointly on a "cycle steal" and interrupt basis. For this reason the card punch needs only 160 microsec of computer time to punch 80 columns, although approximately 500 millisec are required to complete a card punching operation. Because the 160 microsec are consumed on a "cycle steal" basis, the computer continues essentially undisturbed for the 500 millisec, or in this situation, to process two or more soundings during the punching interval. The interrupt from the card punch occurs

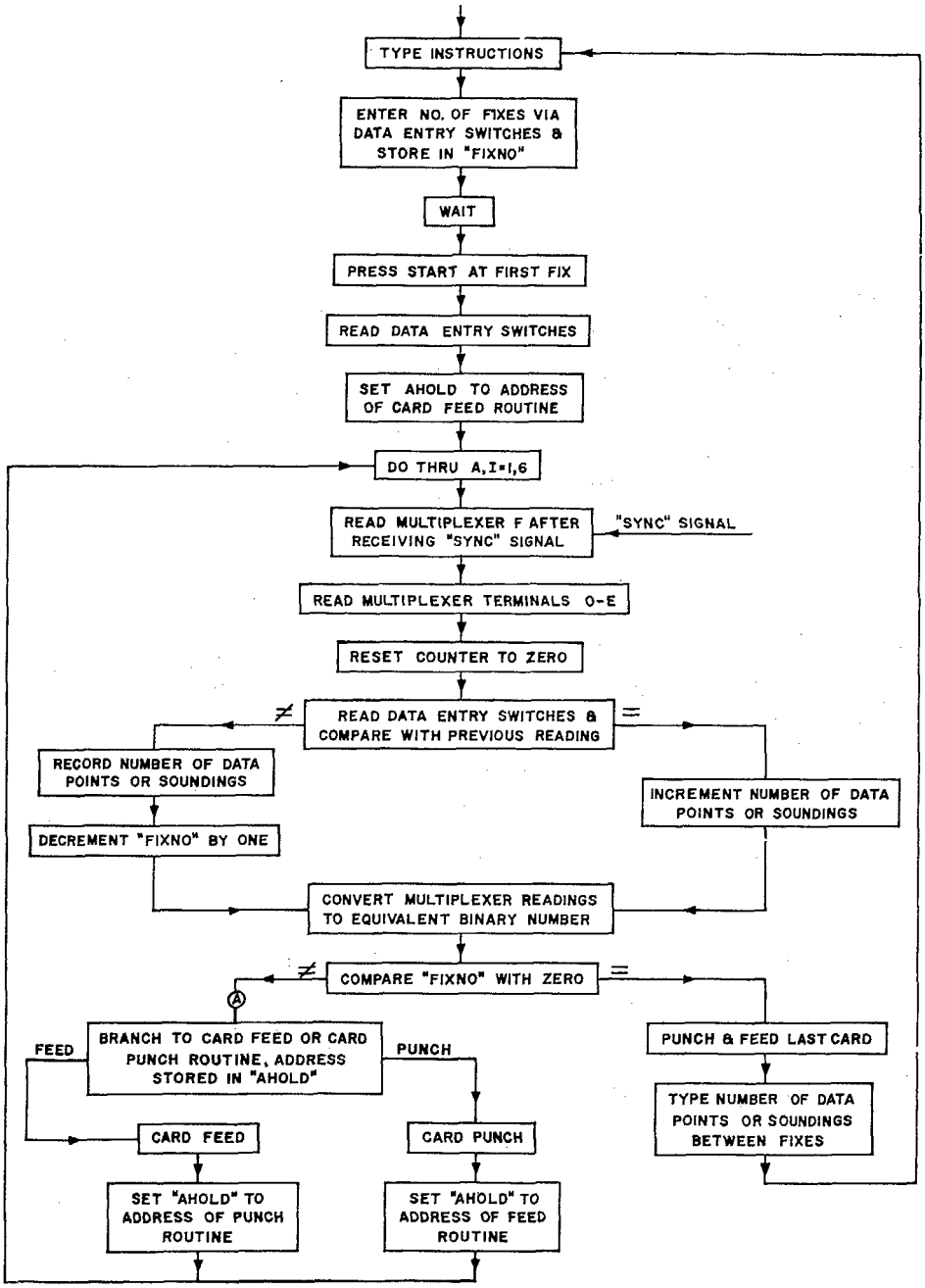


Figure 4 – *Flow chart for the assembler programme*

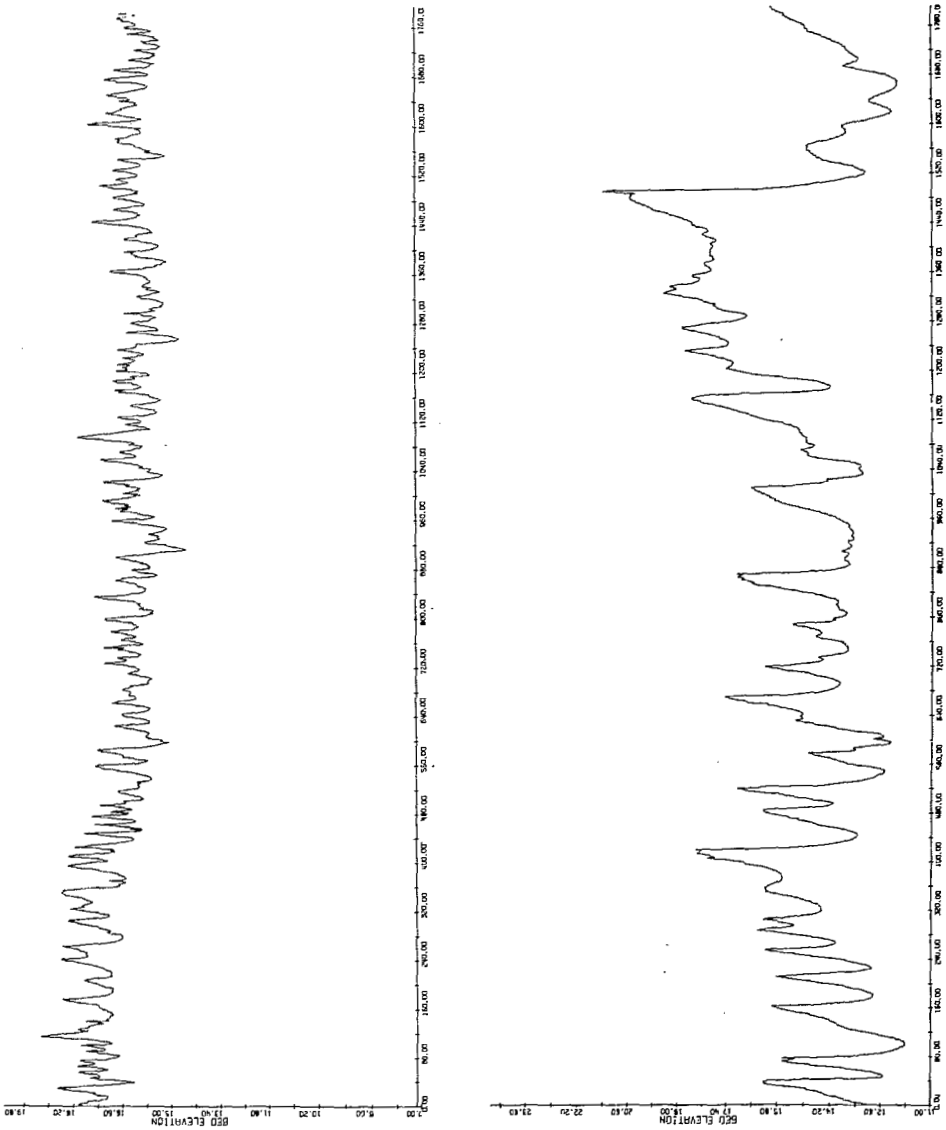


Figure 5 – Typical plots of river-bed profiles

only at the end of the punching interval, i.e. when column 80 has been punched, but because it has a lower priority than the ADC, it does not interfere with the ADC converting process.

RESULTS AND CONCLUSIONS

Figure 5 is a reproduction of computer plots of two typical profiles. The direction of water flow is right to left and the water surface is the abscissa. Vertical resolution is approximately 0.01 ft (0.003 m) and is controlled by the frequency of the oscillator recorded with the sonar signals. The question of resolution was investigated in the laboratory with simulated bed forms in the bottom of the towing tank. These tests showed that the data obtained represented bed profiles quite satisfactorily.

Although the interface circuitry for gating the sonar signals is not included with the transducer, it could easily be integrated with it, and readily adapted to a digital display/recording system. Effective utilization of integrated circuits would provide an instrument suitable for field operation, having the advantages provided by such techniques.

ACKNOWLEDGEMENTS

This programme of investigation was conceived jointly by, and has been carried out under the general supervision of, Mr. Donald C. Bondurant, Missouri River Division of the United States Army Corps of Engineers, and Drs. John F. Kennedy and Emment M. O'Loughlin of the Institute of Hydraulic Research. Grateful acknowledgement is extended to Mr. Robert Livesey of the Corps of Engineers for the co-operation provided in obtaining the loan of a Lockheed Model 417 tape recorder and the Bludworth Sounder. Financial support was provided by the Corps of Engineers, Omaha District, under Contract DACW45-68-C-0055.

BIBLIOGRAPHY/BIBLIOGRAPHIE

Glover, J.R. 1968. Digital acquisition of Missouri River bed profiles. *IIHR Report No. 109*, Institute of Hydraulic Research, University of Iowa, Iowa City, Iowa, U.S.A.

V Measurement of water temperature

Mesure de la température de l'eau

Problème de la mesure de la température de l'eau à l'aide d'un thermomètre

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Résumé. En Tchécoslovaquie, la température de l'eau des rivières est mesurée au moyen d'un thermomètre, une fois par jour seulement, c'est-à-dire le matin au moment de l'observation du niveau de l'eau, aussi loin que possible de la berge et en plein courant, au-dessous du niveau de l'eau, pour une meilleure détermination de la valeur mesurée dans le profil d'un cours d'eau normal.

La température ainsi relevée le matin est inférieure à la température moyenne journalière de l'eau.

L'auteur a comparé cette valeur de la température avec celle de la température moyenne quotidienne, cette dernière étant déterminée à l'aide d'observations effectuées au cours de la journée sur la base de mesures complémentaires réalisées dans les rivières slovaques en 1967.

Les températures mensuelles moyennes de l'eau basées sur les observations faites le matin, de mai à septembre 1967, étaient approximativement les mêmes que les températures moyennes mensuelles mesurées, pour ces mêmes mois, sur une plus longue période. Il est possible d'utiliser les résultats ainsi obtenus pour corriger les observations à long terme effectuées jusqu'à ce jour.

Les températures quotidiennes de l'eau, déterminées comme valeurs moyennes des températures mesurées sur une période de temps limitée, étaient généralement mesurées entre 10 heures et midi et aux alentours de 18 heures. L'écart entre la température quotidienne calculée de la sorte et la moyenne des températures maximale et minimale quotidienne n'atteint en moyenne que 10 % de l'amplitude journalière. Cela démontre que la répartition des valeurs de la température de l'eau au cours d'une journée se fait suivant une courbe symétrique et qu'il est possible de considérer comme température moyenne quotidienne la moyenne des températures extrêmes quotidiennes.

Ainsi il semble souhaitable de recommander que la mesure de la température moyenne des rivières se fasse à l'aide de thermomètres à maxima et à minima.

Cette méthode épargne le recours à des thermographes, appareils dont l'installation est très coûteuse.

PROBLEM OF WATER TEMPERATURE MEASUREMENT IN STREAMS BY THERMOMETER

Abstract. In Czechoslovakia, the temperature of river water is measured by thermometer once daily, in the morning at the same time as the water-level observation. It is taken as far as possible from the bank and in the flowing water below the water-level to give the best determination of the measured value in a normal stream profile.

I compared this value with the daily average temperature of the water determined on the basis of several measurements taken during the day in Slovakian rivers in 1967.

The average monthly temperatures of water from May to September 1967 were approximately the same as values of the average monthly temperatures for these months obtained by taking several readings each day. It is possible to use these results to correct long-term readings taken up to the present time.

The average temperature of water was also determined as the average value of measurements generally taken between 10 and 12 a.m. and at about 6 p.m. The difference between the daily average temperature calculated in this way and that calculated as the average from the maximum and minimum daily water temperature rarely reached an average of 10 per cent of the daily variation. This demonstrates that distribution of the water temperature in the course of a day is symmetric and that it is possible to calculate the average daily water temperature from values of extreme daily water temperatures. Thus, it is perhaps preferable to recommend the measurement of the river temperature with maximum and minimum thermometers.

This method avoids the very expensive installation of thermographs.

PROBLEMAS DE LA MEDICIÓN DE TEMPERATURA DEL AGUA EN CORRIENTES FLUVIALES

Resumen. En Checoslovaquia se mide la temperatura del agua del río una vez al día nada más, con un termómetro, y se realiza únicamente por la mañana al hacer la observación del nivel del agua, tan lejos como se pueda de la orilla y midiendo en una lámina intermedia para mejor determinación del valor medio en el perfil de corriente establecido.

Debido a esta razón, de que dicha temperatura se mida por la mañana, resulta inferior a la temperatura media diurna del agua. La representación de este valor con respecto a la temperatura media diurna del agua y el término de observación adecuado en el curso del día para la determinación de la temperatura media diurna del agua ha sido el tema de mi investigación con base a mediciones politermales de temperatura en los ríos eslovacos, realizadas en 1967.

Las temperaturas medias mensuales del agua a partir de los valores matinales desde mayo a septiembre de 1967 eran aproximadamente los valores de las temperaturas medias mensuales en dichos meses en un periodo extenso, y por tanto los resultados obtenidos son utilizables para la corrección actualizada de los valores medidos hace tiempo.

La temperatura media del agua se determinaba como valor medio de las dos mediciones diarias, por lo general de las 10 a las 12 de la mañana, y hacia las 6 de la tarde. La diferencia entre la temperatura diurna establecida a partir de estos valores y las temperaturas medias diurnas establecidas como media entre los valores máximo y mínimo de la temperatura diurna del agua dan diferencias del orden del 10 por ciento considerando toda la amplitud del intervalo. Este hecho demuestra que la distribución de la temperatura diurna del agua en el curso del día es simétrica y es posible considerar la temperatura media como valor medio entre las temperaturas extremas diurnas ; por tanto, quizá se pueda recomendar la medición de la temperatura del agua del río con termómetros de extrema. De esta forma sería posible obtener el valor de la temperatura de amplitud diaria con termómetros de extrema. Una vez hecho esto, se conseguiría en forma suficientemente exacta la temperature diurna del agua sin instalar termógrafos en los ríos, lo cual resultaría muy costoso.

ПРОБЛЕМА ИЗМЕРЕНИЯ ТЕМПЕРАТУРЫ ВОДЫ В РЕКАХ ТЕРМОМЕТРОМ

Резюме. В Чехословакии температура речной воды измеряется термометром один раз в день, причем делается это утром во время измерения уровня воды, по возможности, дальше от берега и ниже водной поверхности для лучшего определения измеренного значения в постоянном речном профиле.

Вот почему эти температуры, измеренные утром, ниже ежедневных средних температур. Репрезентативность этой оценки по отношению к среднесуточной температуре воды и соответствующего срока наблюдений в течение дня для определения среднесуточной температуры воды были исследованы автором на основе дополнительных многосрочных измерений температуры в реках Словакии в 1967 г. Среднемесячные температуры воды, полученные на основании утренних значений с мая до сентября 1967 г., были приблизительно равны долгосройным значениям среднемесячных температур в эти месяцы, и поэтому полученные результаты можно использовать для исправления измеренных до сих пор долгосрочных значений.

Среднесуточная температура воды определяется, как среднее значение из значений температур воды, измеренных в течение ограниченного времени; обычно она измеряется между 10-12 часами утра и около 6 часов вечера. Разность между суточной температурой, полученной на основании краткосрочных значений температуры воды, и среднесуточной температурой, полученной как средняя из максимальной и минимальной суточных температур воды, достигает в среднем 10 % только от дневной амплитуды. Этот факт показывает, что распределение температуры воды в течение суток является симметричным и среднесуточную температуру воды можно получить как среднее иж экстремных значений суточных температур воды.

Поэтому можно рекомендовать проводить измерение температуры речной воды с помощью экстремальных термометров. Таким образом будет полу-

чено значение суточной амплитуды температур с помощью экстремальных термометров.

Таким путем будет получено значение суточной амплитуды температуры воды, а отсюда — достаточно точное значение среднесуточноч температуры воды, так как установка термографа в реке была бы слишком дорогой.

En Tchécoslovaquie, on enregistre la temperature de l'eau des cours d'eau à l'aide d'un thermomètre à mercure, une fois par jour, dans la matinée, au moment de l'observation de l'échelle limnimétrique, en suivant les "règles concernant la mesure de la température de l'eau dans les cours d'eau [34][1]. La mesure s'effectue le plus loin possible de la rive et dans l'eau courante sous la surface de l'eau. La valeur ainsi obtenue est celle qui caractérise le mieux la température de l'eau dans le profil considéré du cours d'eau.

Toutes ces valeurs sont publiées par l'Institut hydrométéorologique dans les annuaires hydrologiques de Tchécoslovaquie. Comme les mesures sont effectuées dans la matinée, ces valeurs sont inférieures à celles de la température moyenne de l'eau dans la journée.

A la fin du siècle dernier déjà, A. Forster indiquait qu'en mesurant la température dans la matinée, on obtenait la valeur la plus éloignée (dans le sens négatif) de la température moyenne de l'eau pour la journée et que cette valeur tendait asymptotiquement vers la température minimale. Il proposait de mesurer la température quotidienne de l'eau au moment où elle est maximale et au moment où elle est minimale, et de considérer la moyenne de ces valeurs comme étant la température moyenne pour la journée considérée [11].

Matusewicz, considérant les valeurs fournies par le thermographe du Dnestr à Rozwadow en 1931, a constaté que la température quotidienne moyenne de l'eau se situe entre 9 et 12 heures et entre 20 et 24 heures. En ce qui concerne la manière de déterminer cette valeur moyenne, il est du même avis que Forster [30, 31].

Les mesures systématiques effectuées au cours de campagnes spéciales sur la Svir, l'Onega et la Vig inférieure en 1927-1930 ont été exploitées par Vaneeva et Pankrateva [43]. Ces mesures ont confirmé que la température quotidienne moyenne de l'eau se situait à 10-11 heures et à 22 heures.

Wagner, faisant la moyenne des enregistrements thermographiques dans la Werre (dans le secteur de Löhne) en 1950-1951, a déterminé les amplitudes quotidiennes de la température de l'eau (0,3 à 6,2 °C) et l'amplitude annuelle moyenne (2 °C) [44].

En URSS, au cours de la période 1951-1960, diverses campagnes ont procédé à des mesures de la température, en général pendant deux années consécutives, ce qui a permis de déterminer les variations quotidiennes [1, 12, 13, 20, 24]. Ces mesures ont confirmé l'allure symétrique que présente la courbe quotidienne de la température de l'eau de rivière.

Cette symétrie a également été confirmée par les mesures effectuées 24 fois par jour sur le Narew à Pultusk (juillet 1958) et sur la Warta supérieure à Kepowizni (juillet 1959). La valeur moyenne de la température quotidienne de l'eau déterminée comme la moyenne de ces mesures approchait les valeurs de la température de l'eau mesurées à 10-12 heures et à 21-23 heures [46].

En Tchécoslovaquie, les premières mesures destinées à étudier les variations quotidiennes de la température de l'eau de rivière eurent lieu en 1961 à Dalecin sur la Svratka, à l'aide du thermographe Negretti et Zambra.

(1) Les chiffres entre crochets renvoient à la bibliographie en fin d'article.

Nous donnons ci-après quelques résultats de mesures effectuées sur des cours d'eau de la Slovaquie.

Sur l'initiative de la Commission du Danube, on a commencé en 1961 à mesurer la température de l'eau de rivière, y compris dans la soirée, sur le Danube à Komarno, sur le Vah à Sered, et sur le Nitra à Nitrianska Streda (fig. 1).

A Komarno, les mesures furent effectuées à 7 heures et à 19 heures. L'amplitude moyenne mensuelle a atteint son maximum en juillet 1964 avec une valeur de 1,2 °C. Dans les mois estivaux la valeur de l'amplitude moyenne mensuelle se situait dans l'intervalle de 0,5 – 0,7 °C. La moyenne annuelle des amplitudes quotidiennes de la température de l'eau approchait la valeur de 0,5 °C. L'amplitude quotidienne maximale, mesurée le 5 août 1966, atteignait la valeur de 3,4 °C.

A Sered, la température fut également mesurée à 7 heures et à 19 heures. L'amplitude moyenne quotidienne des mois estivaux a atteint la valeur de 1 °C. En juillet 1963, la moyenne des amplitudes quotidiennes atteignait la valeur de 2,3 °C et en août 2,2 °C. Du 11 au 22 août 1963, l'amplitude quotidienne ne descendait pas au-dessous de 2 °C, et du 12 au 14 août elle dépassait la valeur de 5 °C ; le 13 août, l'amplitude atteignait la valeur maximale de 6,0 °C.

A Nitrianska Streda, les observations n'ont pas toujours été faites aux mêmes dates, selon les mois et les années. Dans les mois estivaux, les moyennes mensuelles des amplitudes quotidiennes tendaient vers la valeur de 1 °C. En juillet 1967, cette valeur était de 2,7 °C et en août 1966 elle était de 2,6 °C. L'amplitude quotidienne maximale, déterminée le 27 juillet 1966, atteignait la valeur de 5,5 °C.

De mai à octobre 1967, dans les stations sélectionnées des bassins versants de l'Elbe, de la Morava et du Danube, on a mesuré la température de l'eau 7 fois par jour. En ce qui concerne la Slovaquie, il s'agissait des stations suivantes : Zahorska Ves (Morava), Bratislava (Danube), Komarno (Danube), Sered (Vah) en mai-juin, Sala (Vah) en juillet-octobre, Brehy (Hron), Streda nad Bodrogom (Bodrog).

La température de l'eau était mesurée toutes les deux heures alternativement de 6 heures à 18 heures ou de 7 heures à 19 heures.

Les observateurs qui effectuaient les prélèvements notaient également le temps qu'il faisait durant la journée et les autres paramètres pouvant influencer la température de l'eau dans le profil étudié. Ces mesures ont cependant un inconvénient certain : elles ne sont pas effectuées pendant la totalité du cycle journalier des variations de température. Il serait utile dans ces conditions de faire au minimum des mesures complémentaires de la température de l'eau dans les intervalles de 18-19 à 6-7 heures, en répartissant ces mesures dans toute la largeur du cours d'eau. Parallèlement aux mesures de température de l'eau, il s'avère nécessaire de mesurer également

Tableau 1. *Valeurs caractéristiques des stations thermométriques*

Cours d'eau	Station	Superficie du bassin (en km^2)	Débit moyen annuel (m^3/s)	Température moyenne annuelle de l'eau (°C)
Morava	Zahorska Ves	25 521,3	112	11,2
Danube	Bratislava	131 338,2	1 992	9,7
Danube	Komarno	171 660,4	2 291	10,2
Vah	Sered	10 400,4	140	9,8
Vah	Sala	10 618,9	152	9,5
Hron	Brehy	3 821,4	49,5	8,9
Bodrog	Streda nad Bodrogom	11 311,2	117	9,0



Figure 1 – *Carte du réseau des stations thermométriques*

certains éléments météorologiques. Les résultats de ces mesures représentent pour notre travail une documentation importante.

Pour utiliser scientifiquement les résultats déterminés en 1967, il faut comparer la température moyenne de l'eau dans la matinée, à partir du mois de mai jusqu'au mois d'octobre 1967, avec les valeurs de la température de l'eau prélevées jusqu'à ce jour dans les stations citées.

Les températures moyennes de l'eau dans la matinée étaient, en juin 1967, inférieures à celles du mois de juin observées sur une longue période – sauf sur la Morava à Zahorska Ves et sur le Bodrog à Streda nad Bodrogom. Les températures moyennes de l'eau dans la matinée, aux mois de mai, juillet, août et septembre 1967, tendaient vers celles des mêmes mois sur une longue période. Sauf sur le Hron à Brehy, les températures moyennes de l'eau dans la matinée, en octobre 1967, approchaient les valeurs des températures moyennes mensuelles maximales dans la matinée, au mois d'octobre, sur une longue période.

Il faut cependant noter que sur le Vah à Sala, sur le Hron à Brehy et sur le Bodrog à Streda nad Bodrogom, on ne dispose pour le moment que de courtes séries d'observations.

Les résultats de mesures déterminées (à des dates fixes) sont consignés dans le tableau 2. Pour ce qui est des observations faites à Zahorska Ves aux mois de mai et juin, et à Bratislava, Brehy et Streda nad Bodrogom de mai jusqu'à octobre, il faut considérer comme valables les résultats des mesures effectuées toutes les deux heures entre 6 heures et 18 heures. Pour les observations faites à Zahorska Ves de juillet jusqu'en octobre, à Komarno de mai jusqu'en octobre, et à Sered de mai jusqu'en juillet, il faut considérer comme valables les résultats des mesures effectuées toutes les deux heures entre 7 heures et 19 heures. Dans le tableau, on a consigné la température maximale quotidienne et la température maximale matinale le jour même de la mesure, en 1967. Pour la température maximale matinale, mesurée jusqu'à ce jour, on a mentionné la date (jour et année) de la mesure.

Les symboles utilisés pour ces températures de l'eau (en °C) sont les suivants :

T_Φ température quotidienne moyenne obtenue en faisant la moyenne de 7 valeurs durant le jour,

$T_{\Phi'}$ température quotidienne moyenne obtenue à partir de la température journalière maximale et minimale,

T_R température moyenne matinale.

$T_{10\text{-}11}$, $T_{12\text{-}13}$, $T_{16\text{-}17}$, $T_{18\text{-}19}$: température moyenne mesurée à 10 ou 11 heures, etc.

La température quotidienne moyenne T_Φ était déterminée à partir des mesures (datées) de la température de l'eau. Sauf sur le Bodrog, cette valeur moyenne était trouvée le plus souvent, dans la matinée, entre 10 et 12 heures. Sur le Bodrog, c'était généralement avant 10 heures.

Dans l'après-midi on constatait que la température de l'eau approchait la valeur quotidienne moyenne vers 18 heures sur la Morava et le Bodrog, après 18 heures sur le Danube, le Vah et le Hron. La température quotidienne minimale était enregistrée le plus souvent à 6-7 heures ; aux mois de mai, septembre et octobre, à 8-9 heures.

La température quotidienne maximale était enregistrée tous les mois généralement à 14-15 heures (en juillet et août à 16-17 heures). La différence entre la température quotidienne moyenne et la température matinale moyenne ($T_\Phi - T_R$), sur les secteurs inférieurs des cours d'eau dans les plaines, se limite à une valeur de 1 °C. Sur les petits cours d'eau, cette différence est plus grande.

L'écart entre les deux valeurs moyennes quotidiennes ($T_\Phi - T_{\Phi'}$) a rarement atteint 10 % de l'amplitude quotidienne moyenne, ce qui prouve que la répartition

quotidienne de la température de l'eau est symétrique et on peut déterminer la température quotidienne moyenne en faisant la moyenne des valeurs extrêmes.

Le caractère symétrique de la courbe quotidienne de la température de l'eau de rivière est confirmé par les enregistrements journaliers du thermographe de la Svratka à Dalecin (fig. 2 et 3). Le bassin versant de la Svratka jusqu'à la station de Dalecin, est de 367 km^2 ; le débit annuel est de 3 m^3/s et la température moyenne annuelle de l'eau est de 7 °C. Le bassin versant de la Svratka étant situé dans la Moravie du Nord, nous ne traitons pas des valeurs observées dans ce bassin.

Etant donné que les variations de la température de l'eau de rivière dépendent essentiellement de la radiation solaire et de la température de l'air, la variation de la température de l'eau des cours d'eau pendant la journée est symétrique et elle est approximativement représentée par la fonction :

$$T_{xi} = a \sin (bi + c) + \overline{T}$$

avec :

a = amplitude de la courbe sinusoïdale (en °C); $xi = bi$; $b = \frac{2\pi}{n}$, vitesse angulaire ; pour $n = 12, b = 0{,}52359$; n = nombre de mesures pendant la journée; i = ordre des mesures; c = déviation phasique de la courbe sinusoïdale; T_{xi} = température de l'eau calculée en °C dans le temps i; $\overline{T}$ = température quotidienne moyenne de l'eau (en °C).

L'explication théorique et le développement de la fonction et de l'équation sont indiqués dans une communication antérieure [18].

CONCLUSION

Les exemples mentionnés montrent qu'on ne peut répondre avec précision à la question concernant la représentativité de la valeur matinale de la température de l'eau de rivière en relation avec la température de l'eau journalière moyenne. On ne connaît pas non plus les amplitudes quotidiennes de la température de l'eau de rivière. Il s'avère absolument nécessaire de disposer de ces valeurs, étant donné l'importance que joue l'eau comme refroidissant, notamment dans les usines thermiques.

Dans la bibliographie étudiée, il apparaît que dans la plupart des pays on mesure la température de l'eau une seule fois dans la journée – dans la matinée. Il y a cependant des pays où l'on mesure la température de l'eau de rivière deux fois par jour – dans la matinée – en même temps qu'on procède au relevé des mesures limnimétriques, et dans la soirée vers 18 heures ou vers 20 heures. Il en est ainsi dans plusieurs républiques de l'Union soviétique.

Il y a quarante ans que Matusewicz à la Conférence des Etats baltiques [30, 31], évoquait ce problème qui reste insuffisamment connu et étudié.

Les résultats publiés dans cette communication montrent indiscutablement que, conformément à l'opinion de Matusewicz, on peut déterminer avec précision la température quotidienne moyenne de l'eau de rivière en prenant la moyenne des valeurs extrêmes journalières. Cela évite d'avoir recours à des thermographes, instruments dont l'installation est coûteuse et qui manquent en ce moment sur le marché. Néanmoins il serait souhaitable que de tels appareils soient installés dans quelques stations au moins.

Tableau 2. *Les enregistrements datés de la température de l'eau (°C) en 1967.*

Station	Température maximale en 1967 (et date)		Température maximale mesurée jusqu'à ce jour (et sa date)	Amplitude quotidienne		Température quotidienne moyenne		
	Quotidienne	Matinale		Maximale (et date)	Moyenne	T_ϕ	$T_{\phi'}$	
	Mai							
Zahorska Ves	19,0/16	18,8/17	23,8/25.50	1,9/11-14	0,9	15,7	15,7	
Bratislava	15,0/12	14,8/17	19,9/31.47	2,7/9	1,0	13,1	13,0	
Komarno	15,8/15	15,2/16	20,0/31.47 /27.50	1,7/21	0,9	13,4	13,3	
Sered	17,6/16	16,2/16	21,8/30.11	2,0/20-28	1,3	14,0	13,9	
Brehy	17,6/30	15,4/30	17,4/31.64	4,4/28	2,3	13,3	13,1	
Streda n. Bod.	19,8/16-18	19,6/16-17	19,6/16.67	1,4/4	0,6	16,5	16,4	
	Juin							
Zahorska Ves	23,6/26-30	23,5/29	24,2/19.57	3,0/24	1,0	19,6	19,6	
Bratislava	19,9/30	19,9/30	22,2/4.47	2,0/17	0,9	14,8	14,8	
Komarno	19,9/28-29	18,7/29	23,6/5.47	3,2/3	1,1	15,4	15,3	
Sered	23,0/30	21,7/30	25,7/28.35	2,9/4	1,7	17,2	17,1	
Brehy	23,8/27	21,0/29	22,0/15.64	4,4/23	2,8	16,5	16,4	
Streda n. Bod.	28,4/28	27,0/28	27,0/28.67	1,6/5	0,9	20,2	20,1	
	Juillet							
Zahorska Ves	26,2/2	24,3/24	27,8/31.63	3,0/15	1,8	23,7	23,5	
Bratislava	20,8/23	20,4/23	22,7/9.57	1,8/16-19	0,8	18,8	18,7	
Komarno	21,9/30	20,5/23-24	24,2/5.50	2,2/30	1,1	19,3	19,1	
Sered	25,6/23	23,9/28-29	26,9/1.38	3,4/19	2,0	22,9	22,8	
Sala	26,0/23	23,0/23-24	23,2/26.62	5,3/1	3,1	22,6	22,3	
Brehy	25,4/23	21,2/24	22,2/23.64	5,4/26-27	3,5	21,2	20,9	
Streda n. Bod.	28,2/4	26,8/4.16	27,0/31.64	2,3/19-30	1,5	25,6	25,4	
	Août							
Zahorska Ves	26,6/2	25,3/5	28,7/6.63	3,0/24	1,6	21,9	21,7	
Bratislava	21,4/5	20,8/5	21,7/23.32	2,1/12	1,0	18,4	18,4	
Komarno	21,7/5	20,7/5	24,7/10.51	1,8/16	1,0	18,9	18,9	
Sala	26,2/1	23,5/3	23,5/3.67	1,0/16	2,4	21,2	21,0	
Brehy	25,8/2	21,0/3	22,0/18.62	6,0/16	3,9	20,1	19,7	
Streda n. Bod.	28,0/5	25,8/3	27,2/24.62	2,3/5	1,4	24,1	24,0	
	Septembre							
Zahorska Ves	23,2/7	20,7/4	22,4/1.51	3,2/7	0,9	17,8	17,7	
Bratislava	19,2/11	18,1/4	21,2/5.42	1,6/24	0,8	15,8	15,7	
Komarno	19,9/6	18,5/3,6	24,0/1.50	1,6/5	0,8	16,2	16,1	
Sala	22,0/3-4	19,2/4	19,2/4.67	3,8/3	1,9	17,7	17,7	
Bredy	22,0/7	18,0/10	18,0/10.67	5,2/3	2,8	17,0	16,8	
Streda n. Bod.	26,0/9	24,4/7	26,2/2.62	2,5/4	1,3	22,3	22,1	
	Octobre							
Zahorska Ves	17,3/4	16,8/4	18,6/2.61	0,6/7-26	0,4	13,9	13,9	
Bratislava	17,6/3	16,2/3	17,3/2.32	1,2/24	0,5	13,3	13,3	
Komarno	16,8/4	15,6/4	17,4/1.61	2,4/7	0,9	13,6	13,5	
Sala	18,0/1	15,6/3	16,0/18.66	3,5/1	1,7	13,2	13,2	
Brehy	17,4/13	15,2/5	18,6/27.66	4,2/13	2,2	12,3	12,2	
Streda n. Bod.	21,4/4	20,0/5	20,0/5.67	2,3/3	1,1	16,0	15,9	

Température quotidienne moyenne selon l'heure de la mesure							Ecart entre T_ϕ et les autres températures moyennes					
-7	8-9	10-11	12-13	14-15	16-17	18-19	$T_\phi - T_{\phi'}$	$T_\phi - T_R$	$T_\phi - T_{10\text{-}11}$	$T_\phi - T_{12\text{-}13}$	$T_\phi - T_{16\text{-}17}$	$T_\phi - T_{18\text{-}19}$
,6	15,6	15,6	15,8	15,7	15,8	15,7	0,0	0,1	0,1	− 0,1	− 0,1	0,0
,6	12,8	13,0	13,1	13,3	13,3	13,3	0,1	0,5	0,1	0,0	− 0,2	− 0,2
,9	13,2	13,4	13,6	13,7	13,7	13,4	0,1	0,5	0,0	− 0,2	− 0,3	0,0
,4	13,6	14,0	14,5	14,5	14,2	13,8	0,1	0,6	0,0	− 0,5	− 0,2	0,2
,0	12,9	12,9	13,6	14,0	14,2	13,9	0,2	1,3	0,4	− 0,3	− 0,9	− 0,6
,4	16,5	16,5	16,6	16,6	16,5	16,5	0,1	0,1	0,0	− 0,1	0,0	0,0
,6	19,6	19,6	19,6	20,0	19,6	19,6	0,0	0,0	0,0	0,0	0,0	0,0
,4	14,6	14,7	14,9	15,0	15,1	15,2	0,0	0,4	0,1	− 0,1	− 0,3	− 0,4
,8	14,9	15,4	15,7	15,9	15,8	15,6	0,1	0,6	0,0	− 0,3	− 0,4	− 0,2
,2	16,6	17,2	17,9	17,9	17,6	17,2	0,1	1,0	0,0	− 0,7	− 0,4	0,0
,1	15,5	16,1	17,1	17,5	17,7	17,4	0,1	0,4	0,4	− 0,6	− 1,2	− 0,9
,8	20,0	20,3	20,4	20,4	20,3	20,2	0,1	0,4	− 0,1	− 0,2	− 0,1	0,0
,8	23,2	23,6	24,1	24,4	24,1	23,8	0,2	0,9	0,1	0,4	0,4	− 0,1
,6	18,4	18,7	18,9	19,0	19,0	19,0	0,1	0,4	0,1	− 0,1	− 0,2	− 0,2
,7	19,0	19,3	19,5	19,6	19,6	19,3	0,2	0,6	0,0	− 0,2	− 0,3	0,0
,8	22,3	23,0	23,7	23,8	23,2	22,8	0,1	1,1	− 0,1	0,2	− 0,3	0,1
,9	21,4	22,2	23,1	22,9	23,6	23,2	0,3	1,7	0,4	− 0,5	− 1,0	− 0,6
,2	19,8	20,7	21,9	22,5	22,6	22,1	0,3	2,0	0,5	− 0,7	− 1,4	− 0,9
,7	26,0	25,7	25,9	25,9	25,7	25,5	0,2	0,9	− 0,1	− 0,3	− 0,1	0,1
,0	21,3	21,7	22,1	22,4	22,4	22,1	0,2	0,9	0,2	− 0,2	0,5	− 0,2
,9	18,1	18,3	18,5	18,7	18,8	18,8	0,0	0,5	0,1	− 0,1	− 0,4	− 0,4
,3	18,6	18,9	19,1	19,3	19,3	19,1	0,0	0,6	0,0	− 0,2	− 0,4	− 0,2
,1	20,1	20,3	20,8	21,9	22,0	21,5	0,2	1,1	0,9	0,4	− 0,8	− 0,3
,8	18,3	19,4	21,1	21,6	21,3	20,9	0,4	2,3	0,7	− 1,0	− 1,2	− 0,8
,5	24,0	24,3	24,5	24,4	24,2	24,0	0,1	0,6	− 0,2	− 0,4	− 0,1	0,1
,3	17,5	17,7	17,9	18,2	18,1	17,9	0,1	− 0,5	0,1	− 0,1	− 0,3	− 0,1
,4	15,5	15,7	15,8	15,9	16,0	16,0	0,1	0,4	0,1	0,0	− 0,2	− 0,2
,8	15,9	16,1	16,3	16,4	16,4	16,2	0,1	0,4	0,1	− 0,1	− 0,2	0.0
,0	17,0	17,6	18,1	18,5	18,3	17,9	0,0	0,7	0,1	− 0,4	− 0,6	− 0,2
,5	15,8	16,6	17,6	18,1	17,9	17,4	0,2	1,6	0,4	− 0,6	− 0,9	− 0,4
,7	22,1	22,6	22,8	22,6	22,2	22,9	0,2	0,5	− 0,3	− 0,5	0,1	− 0,4
,7	13,7	13,8	14,0	14,1	14,0	13,9	0,0	0.2	0,1	− 0,1	− 0,1	0,0
,1	13,2	13,3	13,5	13,5	13,5	13,5	0,0	0,2	0,0	− 0,2	− 0,2	− 0,2
,1	13,3	13,5	13,8	13,8	13,7	13,5	0,1	0,5	0,1	− 0,2	− 0,1	0,1
,8	12,6	13,0	13,5	13,9	13,6	13,3	0,0	0,4	0,2	− 0,3	− 0,4	− 0,1
,0	11,3	11,9	12,9	13,2	12,9	12,4	0,1	1,3	0,4	− 0,6	− 0,6	− 0,1
,7	15,8	16,1	16,3	16,2	16,0	15,7	0,1	0,3	−0,1	− 0,3	0,0	0,3

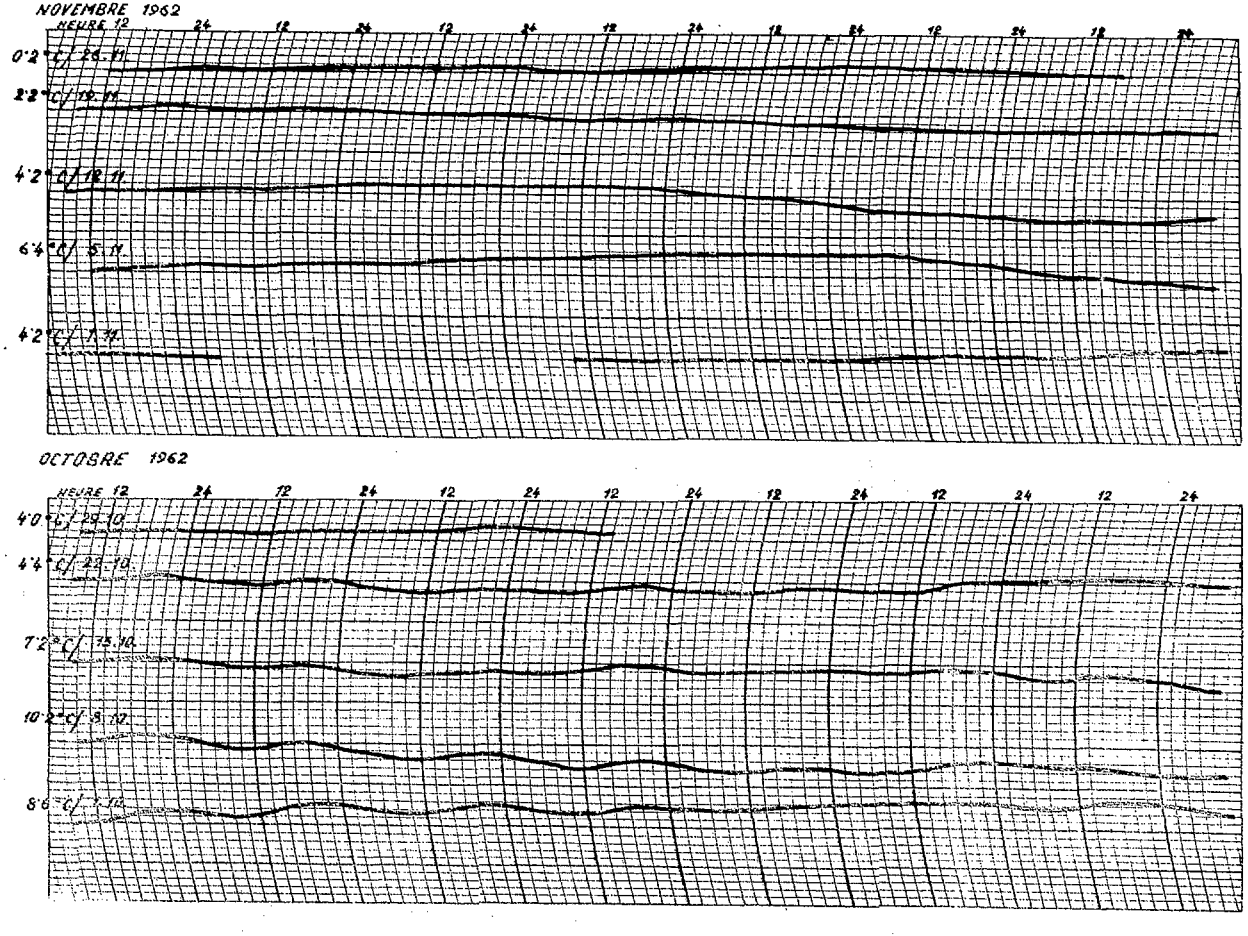

NOVEMBRE 1962
HEURE 12 24 12 24 12 24 12 24 12 24 12 24 12 24
0·2°C/ 26.11
2·2°C/ 19.11
4·2°C/ 12.11
6·4°C/ 5.11
4·2°C/ 1.11
OCTOBRE 1962
HEURE 12 24 12 24 12 24 12 24 12 24 12 24 12 24
4·0°C/ 29.10
4·4°C/ 23.10
7·2°C/ 15.10
10·2°C/ 9.10
8·6°C/ 7.10

Figure 2

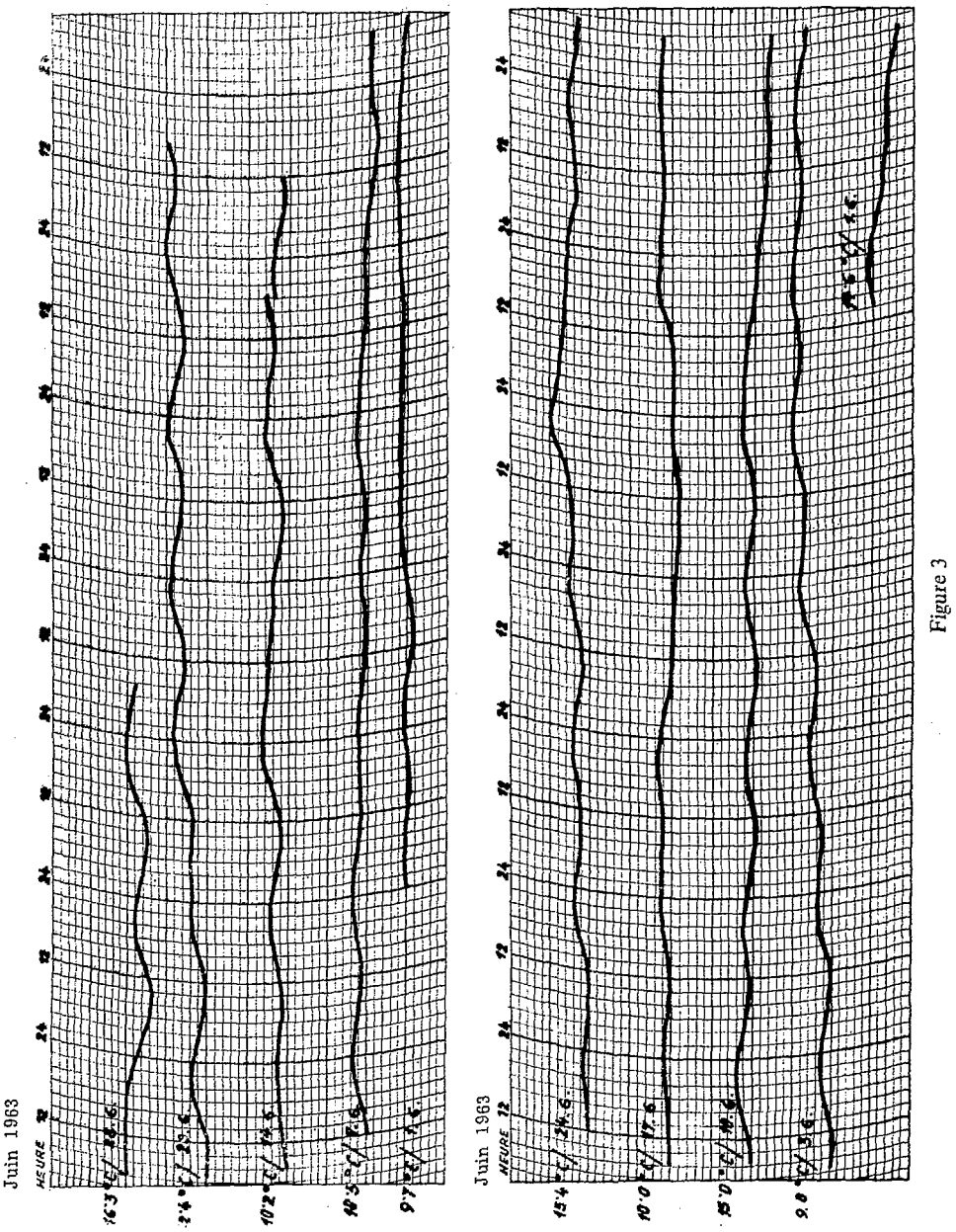


Figure 3

En résumé, on peut recommander que ce problème fasse l'objet de recherches et d'expériences dans le cadre de la Décennie hydrologique internationale.

BIBLIOGRAPHIE/BIBLIOGRAPHY

1. Anosava, L. S. 1956. "Temperaturnyj· režim rek Zapadnoj Dviny (Daugavy)", Izvestija Latvijskoj AN SSR, 8.
2. Bačurin, G. V. ; Vavilova, V. A. 1966. "Temperatura vody i ledovyj režim rek južnoj tajgy Srednej Sibiri", Sb. Klimat i vody juga Srednej Sibiri, Irkutsk.
3. Bydin, F. J. 1938. "Thermal and ice regime of some rivers in USSR and the conditions of foreseeing the same", Association internationale d'hydrologie scientifique, Bulletin n° 23, Riga.
4. Čermák, M. 1959. "Teploty vody na moravských tocích", Praha, HMU.
5. Eckel, O. 1960. "Temperatur der Gewässer in Klimatographie von Osterreich", Denkschr. Akad. Wiss. Bd. 3,2. Lief, Wien.
6. Eckel, 0. 1953. "Zur Thermik der Fliessgewässer. Uber die Aenderungen der Wassertemperatur entlang des Flusslaufs", Wetter und Leben, Sonderheft II.
7. Edinger, J. E. 1968. "The variation of water temperatures due to steam eletric cooling operations", Journal water pollution central federation, 40, no. 9, September.
8. Edington, J. M. 1964. "Some observations on stream temperature", Oikos 15, II.
9. Fal, B. ; Zelazinski, J. 1966. "Zastosowanie termometrów extremalnych do badan nad termika rzek", Prace PIHM, zeszyt 84, Warszawa.
10. Faševskij, B. V. 1966. "O temperature rečnych vod v gorach Altaja", Voprosy geografii Sibiri, Izdatel'stvo Tomskogo univerziteta, sbornik šestoj.
11. Forster, A. 1894. "Die Temperatur fliessender Gewässer Mitteleuropas", Penck's geographische Abhandlungen, Wien, Bd. 5, H. 4.
12. Golovin, V. V. 1965. "Termičeskij reži rek Pamira", Izvestija vsesojuznego geografičeskogo obščestva, 97. n° 2.
13. Golovin, V.V. ; Jarmolinskij, E. A. 1957. "Temperaturnyj režim rek Tadžikistana", Izvestija otdelenija estestvennych nauk Tadžikskoj SSR, vypusk 19.
14. Gutry-Korycka, M. 1967. "Temperature potoków tatrzańskich v róznych źródlach zasilania", Przeglad geograficzny, 39, n° 3, Warszawa.
15. Horváthová, B. 1964. "Teplotný režim slovenskych tokov", Vodohospodársky časopis, 1.
16. Horváthová, B. 1968. "Problematika denných zmien teploty vody tokov", Vodní hospodářství, 7.
17. Horváthová, B. 1969. "Klasifikacia slovenských riek podla teploty vody", Geografický časopis 1.
18. Horváthová, B. ; Dávid, A. 1969. "Ročný ratmus zmien teploty riečnej vody", Vodohospodársky časopis, 2.
19. 1956 – 1965. Hydrologické ročenky ČSSR, I část, Praha, HMÚ.
20. Chmaladze, G. N. 1959. "Termičeskij režim rečnych vod Zakavkazja", Trudy Tbiliskogo naučno-issledovatelskogo gidrometeorologiceskogo instituta, vypusk 5.
21. 1952. Instrukcja dotyczaca pomiarów temperatur wody Cz. 1, Wskazówki dla obserwatorow wodowskazownych, PIHM, Warszawa.
22. 1953. Instrukcja dotyczaca pomiarów temperatury wody, Cz. II. Wskazówki techniczne dla organów nadzorujacych, PIHM, Warszawa.
23. Japa, J. 1938. "Pomiary cieploty wody Popradzie", Acta balneologica II, Kraków.
24. Kisin, J. M., Terterov, A. A. 1959. "Osobennosti termiceskogo rezima rek Dagestanskoj ASSR", Izvestija An Azerb. SSR, serija geol. geogr. nauk.
25. Kříž, V. 1959. "Teplotní režim toku", Vodní hospo arství, 8.
26. Lauscher, F. 1968. "Kurze Messreihen von Gewässertemperaturen und ihre Verwertung", Wetter und Leben, Jhrg. 20.
27. Lászlóffy, W. "Folyóink és tavaink hömérsékleti viszonyai", Beszamoló Vituki, Budapest.
28. Macan, T. T. 1959. "The temperature of a small stony stream", Hydrobiologia, acta hydrobiologica, hydrographica et protistologica, Haag 12.
29. Malankina, J. P. 1962. "Osnovnye charakteristiki termičeskogo režima rek BSSR", Sbornik rabot Minskoj gidrometeorolog. observatorii, vypusk 4.
30. Matusewicz, J. 1930. "Betrachtungen über die Methodik der Untersuchungen der Temperatur fliessender Gewässer", III. Hydrolog. K.d.b.s., Warszawa.

31. Matusewicz, J. 1933. "Die Arbeitsmethoden betreffend die Untersuchungen der Temperatur fliessender Gewässer", IV Hydrologische Konferenz d.b.s.
32. Musaeljan, S. M. 1968. "Nekotorye voprosy termičeskogo režima rek bassejna Araksa", Sbornik rabot po gidrologii, n° 8. Gidrometeoizdat, Leningrad.
33. Nagawiecka, H. 1966. "Termika wody potoków górskich w Jaworkach kolo Szczawnicy", Roczniki nauk rolnicznych, tom 118-D, Warszawa.
34. 1958. Návod na meranie teploty vody vodných tokov, HMU, Praha.
35. Nishizawa, T. 1969. "Introduction to river water temperature", A. Part of contemporary to climatological papers.
36. Petkov, A. 1960. "Temperaturen režim na rekite v NR Bulgarija", Chidrologija i meteorologija, 5.
37. Plaščev, A. V. 1966. "Termičeskij režim rek Kryma i vlijanie na nego karstovogo pitanija", Trudy Ukr. n-i., gidromet. inst., vyp. 60.
38. Pleskot, G. 1951. "Wassertemperaturen und Leben im Bach", Wetter und Leben, 3, Heft 5-7.
39. Rymša, V. A. 1959. "Ledovye issledovanija na rekach i vodochraniliščach", Gidrometeorologičeskoe izdatelstvo, Leningrad.
40. Sauber, F. 1951. "Registrierungen der Temperatur der Fliessgewässer", Wetter und Leben, 3, Heft 9-10.
41. Smith, K. 1968. "Some thermal characteristics of two rivers in the Pennine area of Northern England", Journal Hydr. 6.
42. Sommani, E. 1965. "Osservazioni sulla termica dei corsi d'aqua alpini", Boll. Pesca, Piscicolt. e Idrobiol., 20, n° 2.
43. Vaneeva, C. V., Pankrateva, M. N. 1941. "Issledovanije termičeskogo režima rek Sviri, Onegi i nižnego Viga", Trudy GGI, vypusk 11.
44. Wagner, H. 1952. "Die Verwendung von Thermographen in fliessenden Gewässern", Denksch. der Gewässerkundlichen Dienststelle Jahrbuch Bielefeld, n° 4.
45. Wytyczne dla obserwacji extremalnych temperatur wody, PIHM (strojopis), Warszawa.
46. Zelazinski, J. 1963. "Dobowy przebieg temperatury wody W rezkach", Gos podarka wodna, XXI, 3.
47. Yakuwa, J. "The longitudinal turbulent motion of a river flow and its role on the diurnal variation of water temperature".

The measurement of mean temperature on a reaction velocity basis and its application to hydrology

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Abstract. The dependence of the reaction velocity of sucrose hydrolysis on temperature has been used for many years to estimate the temperature of the surroundings of exposed glass tubes containing defined sucrose solutions. The changes in concentrations of sucrose during the exposures, measured in a polarimeter, are the basis for calculations of average temperatures. The method has been improved and is now very reliable, so that it has been applied to many programmes of measuring water temperatures, especially to control the influence of cooling water discharges of power stations. The method is described, including details of the exposure technique and computer programmes for data calculations. The results of temperature studies in the River Neckar are discussed.

MESURE D'UNE TEMPERATURE MOYENNE A PARTIR D'UNE VITESSE DE REACTION ; APPLICATION EN HYDROLOGIE

Résumé. La vitesse de réaction de l'hydrolyse du sucre de canne étant fonction de la température, on utilise depuis de longues années ce phénomène pour mesurer la température d'un milieu où sont placés des tubes de verre contenant une solution de sucre de canne bien définie. Les variations de la concentration du sucre de canne durant le temps de l'exposition sont mesurées au polarimètre et servent de base au calcul des températures moyennes. Cette méthode, qui a bénéficié de diverses améliorations, est très sûre, de sorte qu'elle a pu s'insérer dans nombre de programmes de mesure de la température de l'eau, notamment pour contrôler l'influence des déversements d'eau de refroidissement dans les centrales nucléaires. La méthode est décrite avec tous les détails concernant le mode d'exposition et les calculs sur ordinateur. L'auteur traite de résultats obtenus sur le Neckar.

LA MEDIDA DE LA TEMPERATURA MEDIA A PARTIR DE LA VELOCIDAD DE LA REACCIÓN DE HIDRÓLISIS Y SU APPLICACIÓN A LA HIDROLOGÍA

Resumen. Al ser función de la temperatura la velocidad de la reacción de hidrólisis del azúcar de caña, se utiliza desde hace muchos años este fenómeno para medir la temperatura ambiente depositando tubos de vidrio, que contienen una disolución de azúcar de caña bien definida. Las variaciones en la concentración del azúcar durante el tiempo de exposición se miden con un polarímetro y sirven de base para el cálculo de las temperaturas medias. El método, mejorado a lo largo del tiempo, es muy seguro, de forma que se ha utilizado en muchos programas de medida de la temperatura del agua, en especial para controlar la influencia de los vertidos de agua de refrigeración de las centrales nucleares. El método se describe con todos los detalles relativos al modo de exposición y a los programas necesarios para el cálculo de datos por ordenador electrónico. El autor expone los resultados de los estudios relativos a la temperatura realizados en el río Neckar.

СПОСОБ ИЗМЕРЕНИЯ СРЕДНЕЙ ТЕМПЕРАТУРЫ ПО СКОРОСТИ РЕАКЦИИ ОСНОВАНИЯ И ПРИМЕНЕНИЕ В ГИДРОЛОГИИ

Резюме. Зависимость скорости реакции гидролиза сахарозы от температуры использовалась в течение многих лет для определения температуры внешней среды экспонированных стеклянных трубок, содержащих определенные растворы сахарозы. Наблюденные изменения в концентрациях сахарозы в период выдержек, измеренные в поляриметре, являлись основой для вычислений средних температур. Этот метод затем был улучшен и является теперь вполне надежным; поэтому он был применен во многих программах, включающих измерение температур воды, особенно для контроля влияния за охлаждением воды, пропускаемой через гидростанции.

Данный метод описан, включая детали методики экспозиции и составления программ для обработки данных компьютером. Приводятся результаты температурных исследований по р. Неккар.

BASIS OF THE METHOD

The method of temperature measurement described uses the dependence of the rate of sucrose hydrolysis on temperature to obtain data from which the average environmental temperature of a sucrose solution during a definite time of exposure may be calculated.

According to the law of mass action, the chemical reaction velocity (RG) depends on the concentration of the reacting substance in the solution. Since this concentration changes continuously during the reaction, the reaction velocity also changes constantly. Independent of the concentration conditions, the reaction velocity is defined by means of the momentary velocity constant k. For a unimolecular, nonreversible reaction of the type

$$YZ \rightarrow Y + Z$$

the velocity constant is

$$k = \frac{1}{t} \cdot \log \frac{A}{A - x}$$

where A is the initial concentration of the substance YZ, and $A - x$ is the concentration after a time t.

Principally, the hydrolysis of sucrose

$$\underset{\text{(sucrose)}}{C_{12}H_{22}O_{11}} + H_2O \longrightarrow \underset{\text{(D-glucose)}}{C_6H_{12}O_6} + \underset{\text{(D–(–)fructose)}}{C_6H_{12}O_6}$$

may be treated as a unimolecular reaction because, in aqueous solutions, the concentration of water may be considered constant. Since we are concerned with optically active reaction material, the concentration ratio

$$\frac{A}{A - x}$$

may be replaced by the optical rotation angle ratio

$$\frac{R_0 - R_\infty}{R_t - R_\infty}$$

where R_0 is the optical rotation angle of the sucrose solution at the beginning of the reaction time t, R_t the optical rotation of the solution after the reaction time t, and R_∞ the optical rotation after the completion of the reaction. Hence, the reaction velocity of the sucrose hydrolysis can be defined by the velocity constant

$$k = \frac{1}{[H^+] \cdot t} \cdot \log \frac{R_0 - R_\infty}{R_t - R_\infty}. \tag{1}$$

The reaction is catalyzed by hydrogen ions, so that the reaction velocity is proportional to the hydrogen ion concentration $[H^+]$; this is taken into account in equation 1.

In chemical reactions, the dependence of reaction velocity on the temperature is described by the Arrhenius equation

$$\log k = b - \frac{a}{T\,[°K]}$$

in which the logarithm of the velocity constant changes linearly with the reciprocal value of the absolute temperature, T. Thus, for sucrose hydrolysis:

$$\log \left[\frac{1}{[H^+] \cdot t} \cdot \log \frac{R_0 - R_\infty}{R_t - R_\infty} \right] = b - \frac{a}{T}$$

And since $\log \frac{1}{[H^+]} = \text{pH}$

$$T = - \frac{a}{\text{pH} - b - \log t + \log [\log (R_0 - R_\infty) - \log (R_t - R_\infty)]}. \tag{2}$$

Accordingly, one can compute the reaction temperature of sucrose hydrolysis if the pH of the solution, the reaction time, t, and the rotation angles R_0, R_t, and R_∞, are known, as well as the constants a and b. The latter can be determined experimentally for given values of T, pH, and t, by measurement of R_0, R_t, and R_∞. According to the investigations of Pallmann *et al.* (1940) the value of the constants for the sucrose solution (preparation : see below) are $a = 5\,856.6$ and $b = 20.1998$.

Therefore, as Pallmann *et al.* (1940) first suggested, it is possible to use an acidified sucrose solution in thermometry. If such a solution is exposed over a given time period t, and R_0, and R_t, and R_∞ are measured, the mean temperature to which the solution was exposed during the exposure time can be computed.

Details of the technique of measurements were reported in several publications (Schmitz and Volkert, 1959, 1966; Schmitz, 1964, 1966). For the measurements a sucrose buffer solution is used, which is prepared by dissolving 1,500g sucrose in 1,000 ml distilled water and mixing with a citrate buffer solution (404 ml 0.2 n sodium citrate solution + 596 ml 0.2 n HCl) in the volume ratio 1 : 1. To prevent growth of micro-organisms, this solution also contains 0.35 per cent formaldehyde.

The optical rotation angles are measured with an electrical polarimeter or with a visual circular-scale polarimeter, the precision of which should be ± 0.005°. Measurement is best with spectrally selected green mercury light ($\lambda = 546.1$ nm) and less good with yellow sodium light ($\lambda = 589.3$ nm) The pH of the sucrose solution is 3.89, but it may be lowered by the addition of concentrated HCl to pH 1.2 without

leaving the range where equation 2 is valid. In this way, the relationship between the variation of rotation angle and the temperature during exposure stays in a well measurable range. Graphical representations were published by Schmitz and Volkert (1959, 1966).

For exposure, the sucrose solution is poured into thick-walled Pyrex tubes which are closed by a tight polyethylene stopper or screw cap. They are waterproof and may be inserted—preferably in a perforated plastic protection tube—directly into the water. The tubes are transported to the exposure locations in an ice-sodium chloride mixture to prevent inversion of sucrose during transport.

The evaluation of measurements can be simplified if, for one or several series of measurements, a large amount of sucrose solution is prepared at once. In this way data for $[H^+]$ and R_0 are constant, and the precision of the resulting data is increased. Results from identical solutions are exactly comparable. Deviations in the accuracy of measurements of pH influence the calculations of temperature data. For temperature data accurate to $\pm 0.1\,^{\circ}C$ an accuracy of $\pm 0.01 - 0.02$ pH units is required. The solution may be stored indefinitely at low temperature. It freezes at $-4{,}7^{\circ}C$, at which temperature no inversion takes place. But even at $0^{\circ}C$, the velocity of inversion is so slight that if pH is not too low and storage time not too long only slight inversion occurs, so that the solution can be used after measuring R_0.

Computer programmes and tables are available to perform the necessary calculations of T from equation 2.

According to equation 2, the mean temperature measured over the exposure time t is not identical with the arithmetical average of temperature during the exposure time. The temperature T measured by the sugar inversion method represents an exponential mean, which may be termed "effective average temperature", T_{eff}. According to the mathematical meaning of the value T_{eff} (mathematical derivation by Schmitz and Volkert, 1959, 1966), T_{eff} measured by the inversion method is always somewhat higher than the arithmetical average $\bar{T}$. However, it can be shown (Schmitz, 1954) that measuring water temperatures with variations of only a few $^{\circ}C$ during a reasonable period of time, T_{eff} deviates from $\bar{T}$ only by a few tenths of $1^{\circ}C$. Thus this method is also useful for studying problems of water temperatures of heat budgets in waters. This may be demonstrated by a few examples.

EXAMPLES OF RESULTS

1. The summer temperature of streams in central Europe. Hydrological and hydrobiological problems stimulated a comparative study of average summer temperatures of different streams in central Europe in July and August 1967. A total of 120 measuring locations were used. Figure 1 demonstrates the summer temperatures in a selection of streams. The increase in the temperature of a stream during summer is proportional to the logarithm of the distance from its origin, as long as it is below the equilibrium of the heat budget. Furthermore, the initial temperature in the headwater area is important for the formation of a certain temperature zone in the course of the stream. If this is very low, zones of moderate summer temperatures of about $13\text{-}16^{\circ}C$ extend downstream for a long way. At high initial temperatures such a zone has only a short length.

Figure 2 is a schematic map and diagram of temperatures of a number of streams between Denmark and northern Italy, based on the measurements in summer 1957.

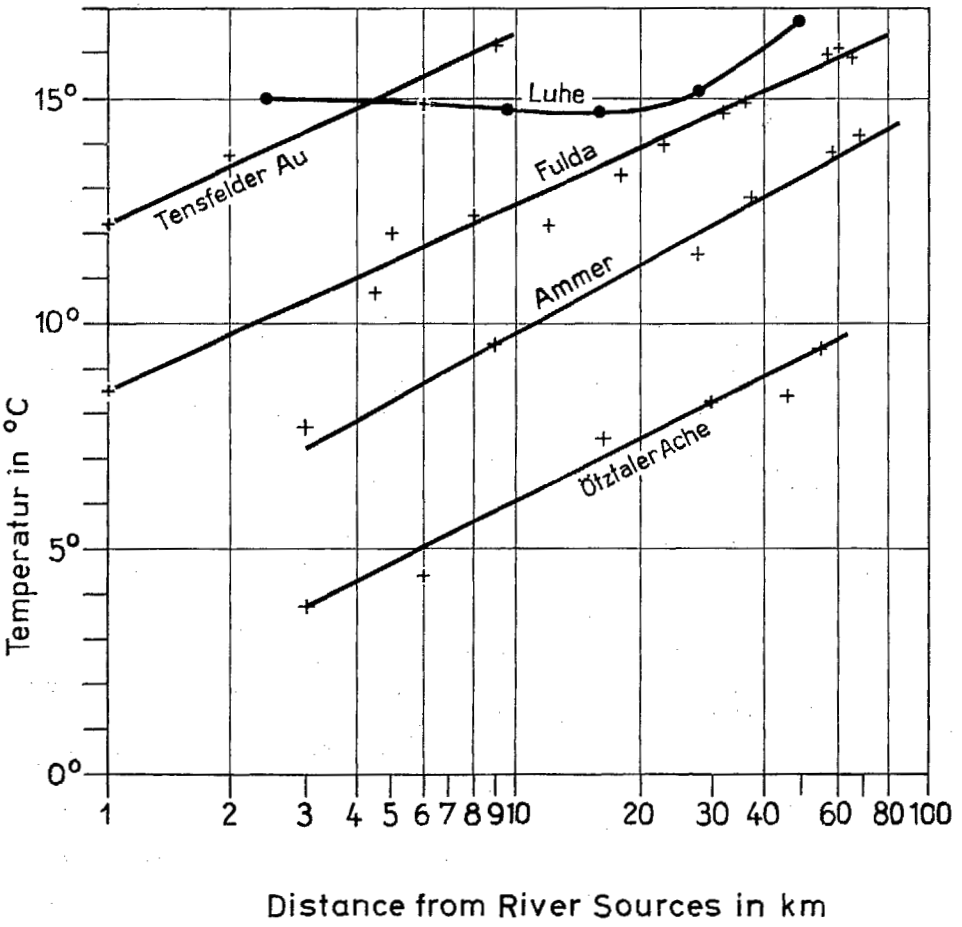


Figure 1 – *Relation between water temperature and distance from river sources.*

The streams are arranged according to geographical latitude, which results in a longitudinal transect through central Europe. Positions of equal summer temperatures of these streams are connected by isotherms.

2. Summer temperatures in Yellowstone Park streams. Figure 3 shows the average summer temperatures in Firehole and Madison River in the Yellowstone Park. Here the usual situation is reversed. At their origin the waters are warmed by hot geyser effluents and by thermal springs. In summer, therefore, the stream temperatures are above the equilibrium of the heat budget under the high altitude climate of the Yellowstone Park. With increasing distance from their origin, therefore, streams of this area become cooler in summer.

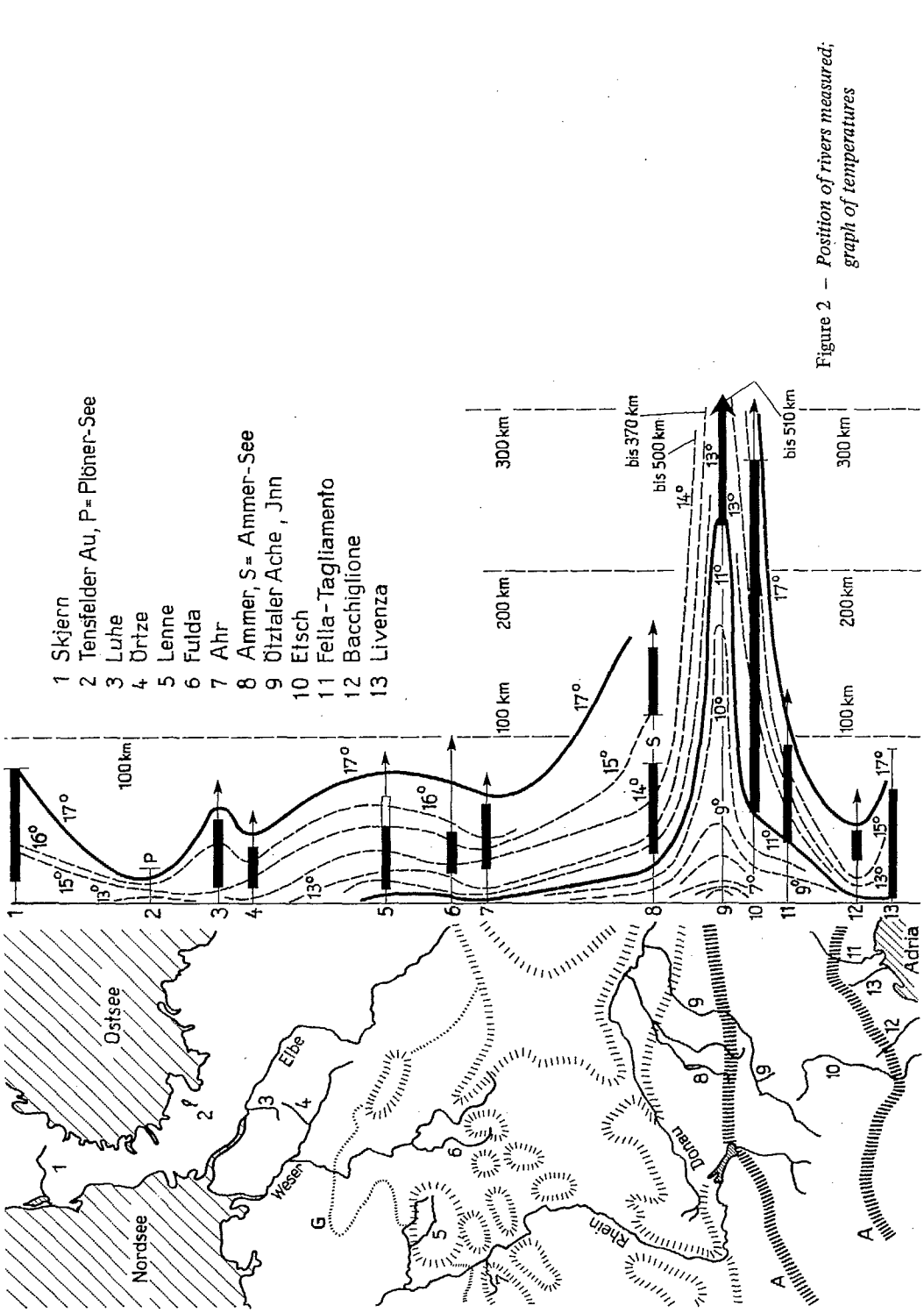


Figure 2 – *Position of rivers measured; graph of temperatures*

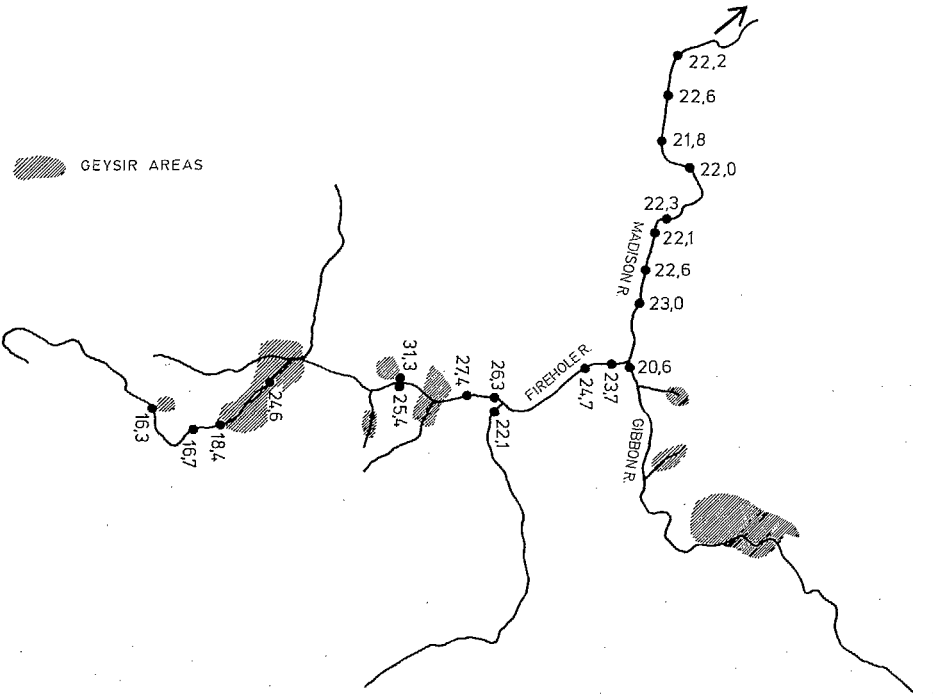


Figure 3 – *Average summer temperatures in streams of Yellowstone Park*

3. Temperature records from the River Neckar Figure 4 represents a review of temperature conditions of the lower Neckar. In this case we can recognize conditions of local warming, which are followed by ranges where the temperature becomes lower again. The increase of temperature comes from the release of cooling water from power plants. In so far as this warming leads to temperatures which are above the equilibrium of the heat budget of the stream, a cooling will follow downstream until equilibrium has again been established. In other parts of the Neckar, lower temperatures occur because of the influence of colder tributary streams.

4. Temperature measurements in stratified waters. The average temperature of different water layers of standing waters may be measured by the sugar inversion method. The temperature tubes are fastened on a string, which extends between a ground anchor and a surface buoy. In this way temperatures are now being measured in the littoral of the Bodensee Untersee.

These few examples demonstrate that temperature measurements based on the velocity of reaction may give useful results in limnological work and applied problems. This method is particularly useful if simultaneous measurements must be made at numerous locations ; the alternative technique would be the installation of very expensive recording instruments.

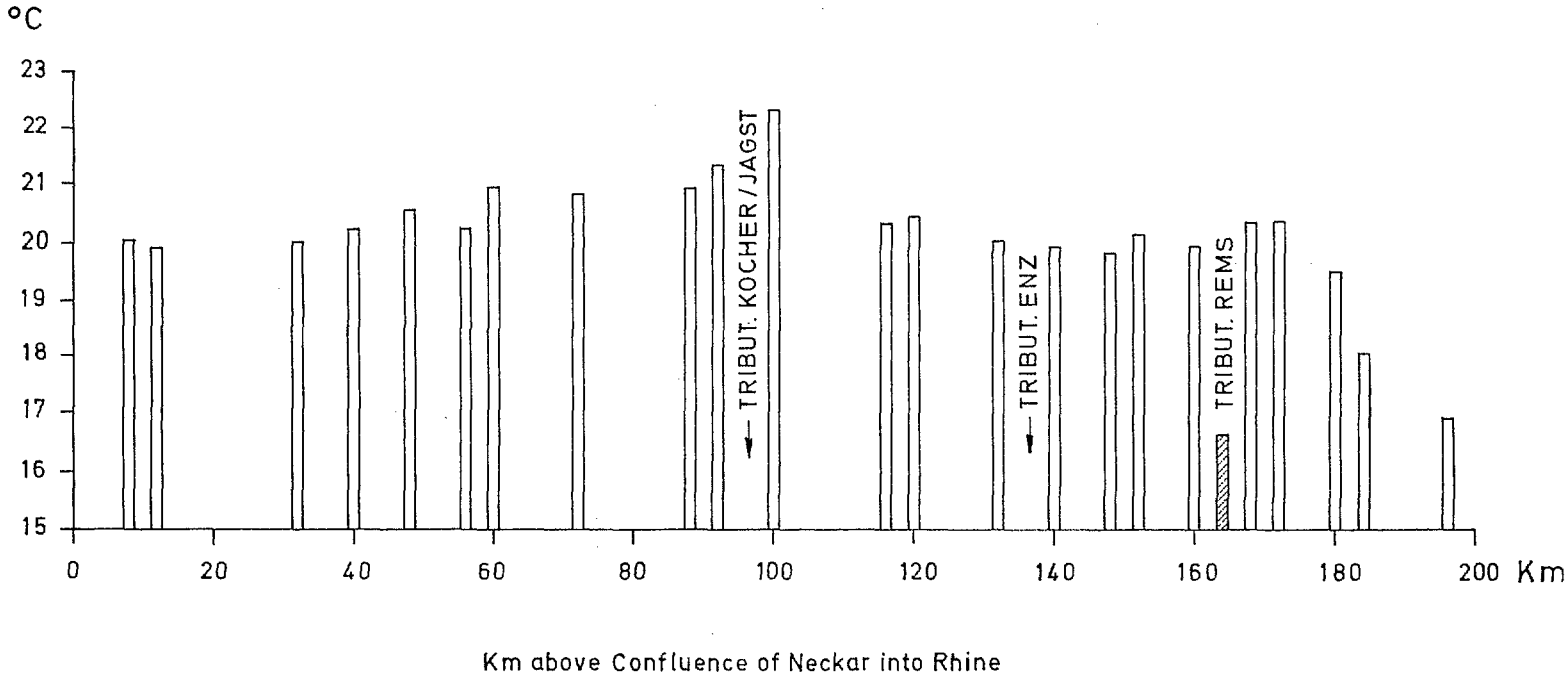


Figure 4 – *Average temperature in the River Neckar, 29 August to 15 September 1967*

BIBLIOGRAPHY / BIBLIOGRAPHIE

Pallmann, H.; Eichenberger, E.; Hasler, A. 1940. Eine neue Methode der Temperaturmessung bei ökologischen und bodenkundlichen Untersuchungen. *Ber. Schweiz. Bot. Ges.*, vol. 50, p. 337-62.

Schmitz, W. 1954. Grundlagen der Untersuchung der Temperaturverhältnisse in den Fliess gewässern. *Ber. Limnol. Flusstation Freudenthal*, vol. 6, p. 29-50.

———. 1964. Messprobleme bei reaktionskinetischen Untersuchungen mit dem Kreispolarimeter, dargestellt am Beispiel der reaktionskinetischen Temperaturmessung. *Zeiss-Mitt*, vol. 3, no. 6, p. 227-49.

———. 1966. Measurement Problems in Reaction-Velocity Investigations with the Polarimeter, as exemplified by Reaction-Velocity Temperature Measurement. Translation, U.S. Department of Commerce, Environmental Science Services Administration, Environmental Data Service. WB/T-112.

Volkert, E. 1959. Die Messung von Mitteltemperaturen auf reaktionskinetischer Grundlage mit dem Kreispolarimeter und ihre Anwendung in Klimatologie und Bioökologie, speziell in Forst- und Gewässerkunde. *Zeiss-Mitt*, vol. 1, no. 8/9 p. 300-37.

———. 1966. The Measurement of Mean Temperature on a Reaction Velocity Basis with the Polarimeter and its Use in Climatology and Ecology—Especially in Hydrology and Forestry. Translation, U.S. Department of Commerce,, Environmental Science Services Administration, Environmental Data Service, WB/T-110.

DISCUSSION

Grindley: Mr. Schmitz has given us an interesting and erudite paper bringing in several sciences such as chemistry, physics, physical chemistry and also computer techniques. Could he please tell me what is the advantage of using such a complicated method for determining temperature?

Schmitz: The main advantage of this method is its cheapness. Fluctuations of temperature occurring daily in a stream require comparative studies using equipment which permanently records these fluctuations. The 120 measuring stations existing in central Europe would need expensive equipment; maintenance and observation are also expensive. This method, however, is very simple. A test tube with a protective cover is placed into a stream for a while, the probe is collected and the rotation angles are measured normally in the laboratory.

Burz: The advantage which you describe is not convincing since conventional measurement with a thermometer is really simple.

Schmitz: The measurement with a thermometer is good and sufficient if no fluctuation of temperature occurs during the interval between the two measurements. Unfortunately, such fluctuations do occur and require either more frequent measuring or a recorder. In the case of some major rivers one measurement daily is sufficient, but this is certainly not so in other cases.

Burz: My second question: how do you define the mean temperature of summer? Is it a mean temperature established with time units or a defined value?

Schmitz: The word "summer" was only a general expression. For our investigations in Denmark and in Italy we limited the time to the months of July and August. In the Yellowstone National Park we worked with monthly means. This method permits the

time periods to be established according to the needs. If the mean temperature for 24 hr is requested, the solution would be made more acid in order to accelerate the reaction velocity.

Burz: A third point: the thermic longitudinal section in a river may vary considerably due to inflow and losses.

Schmitz: That is correct, but it would take too long to discuss these influences here.

Burz: A fourth question: did you also make measurements in lakes?

Schmitz: Yes. Studies are being carried out in the Bodensee. The tubes are fixed to a string which is fastened between a buoy and an anchor. We obtain average temperatures in different depths in a vertical line.

Gruber: What is the maximum period for a measurement?

Schmitz: Theoretically measurements are possible until the solution is exhausted. However, in practice the limit should be drawn at 80 per cent so as not to influence the accuracy of the results. In fact, this problem does not normally occur, since, according to our experience, periods longer than two weeks are not very practicable. We have also measured for periods of two months, but I believe that it is not very useful to measure for longer periods since this results in a lack of data.

Gruber: A second question: is the method also applicable during low temperatures?

Schmitz: During low temperatures the reaction velocity is very slow; I mentioned that we preserve the solution at 0°C. But if the solution is made very acid (ph = 2) results can be obtained. However, we normally apply in the cold season periods of one month.

Keller: Did you apply overlapping measurements?

Schmitz: No, for reasons of manpower and costs.

Keller: In one of our small mountain rivers we applied this method. The probes were changed every second week, with an overlap of one week.

VI Measurement of solid matter transport

Mesure des transports solides

Experiences with photometric turbidity measurements

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Abstract. Within the range of the extensive sediment measurements carried out in the Bavarian rivers, two turbidimeters (Sigrist photometer and Askania turbidimeter) have been tested during the past two years. In the course of these tests special emphasis was placed on their reliability in operation, maintenance and expenditures and calibration.

The first part of the paper deals with general questions of photometric turbidity measurements; the second part provides a short description of the instruments tested with some detailed data as to their design and mode of operation. Finally, a discussion of the measurements and calibration results obtained is presented.

EXPERIENCES DE MESURE DE LA TURBIDITE PAR PHOTOMETRIE

Résumé. Dans le cadre des études extensives des matériaux en suspension dans les rivières bavaroises, deux turbidimètres (turbidimètre Sigrist et turbidimètre Askania) ont été mis à l'essai au cours des deux années écoulées.

Au cours de ces expériences, on a prêté une attention particulière à la fiabilité des appareils, aux frais de leur entretien et à leur étalonnage. La première partie de ce rapport donne des informations générales sur les mesures photométriques de la turbidité. La deuxième partie décrit les appareils étudiés et fournit quelques données sur leur construction et leur fonctionnement. Enfin, les résultats des mesures et de l'étalonnage font l'objet d'un examen critique.

EXPERIENCIAS FOTOMÉTRICAS PARA MÉDICION DE TURBIDEZ

Resumen. Dentro del marco de las mediciones extensivas de sedimentos realizadas en los ríos de Baviera, se han ensayado dos turbidímetros (Fotómetro Sigrist y Turbidímetro Askania) que se habían ensayado en los dos últimos años. En el curso de estos ensayos se puso particular atención primordialmente en la fiabilidad de funcionamiento, exigencias de mantenimiento y calibración.

La primera parte de la presente comunicación trata de temas generales de las mediciones de turbidez fotométrica ; la segunda parte del mismo da una breve descripción de los instrumentos ensayados con algunos datos detallados respecto a su diseño y forma de funcionamiento. Finalmente, se discuten las mediciones y resultados de calibración obtenidos.

ОПЫТ ИЗМЕРЕНИЯ МУТНОСТИ С ПОМОЩЬЮ ФОТОМЕТРА

Резюме. В целях более широкого охвата измерениями наносов на реках Баварии в течение двух последних лет испытывалось качество двух батометров (фотометра Sigrist и батометра Askania). Основное внимание во время этих испытаний уделялось надежности этих приборов в работе, уходу за ними, а также расходам по содержанию и тарировке.

Первая часть этой статьи посвящена общим вопросам фотометрических измерений мутности, вторая часть содержит краткое описание испытываемых приборов с детальными данными в отношении их конструкции и способов использования. В конце доклада рассмотрены полученные результаты измерений и тарировки.

INTRODUCTION

For continuous measurements of suspended load in rivers and for the control of water by means of photometric turbidity measurements, exact reproducibility is required, as well as high sensitivity and accuracy. Furthermore, instruments must be highly reliable and require a minimum of maintenance. For several years, commercial instruments have been tested at various control and gauging stations. The dominant features in these tests have been the calibration of the turbidity measurements and the discussion of the reference units.

The author's tests were carried out with the Sigrist photometer and Askania turbidimeter. For calibration, use was made of artificial suspensions of weighed quantities of sediments as well as of natural suspensions and of river water. The measurements were executed in the River Main near Markbreit and in the River Isar near Munich. The measuring systems of both instruments will be discussed briefly. The installation and the mode of operation of the devices concerned cannot be dealt with here, but are described in the references.

MEASURING SYSTEM

The Sigrist photometer is a two-beam apparatus designed on the fully automatic optical compensation system. It is based on the reflection method in which the amount of scattered light is used as a measure of the turbidity. In this method all particles of a size exceeding the wave length of light (approximately 0.5 μ) will be included.

Both the measuring and the comparative beam are from the same lamp, and are measured in the same photo-cell. Measurements are carried out according to the zero method, i.e. an automatically adjustable shutter shuts off the comparative beam until the photo-cell received light of equal intensity from both the measuring and comparative beams. This type of measurement combines high reliability with high accuracy.

The light beams arriving at the common photo-cell produce—at different intensities—a photo-current, which is amplified. As soon as equal light intensity is achieved, the alternating impulse stops and the required value can be measured. This system eliminates any extraneous factors affecting the measurements such as fluctuations of current voltage, differences in the intensity of the lamp, and variations in the degree of amplification. Thus the measured value is controlled only by the structure of the light beam and the regulating shutter.

The measuring range and the sensitivity can be varied within considerable limits by the replacement of the turbidity standards. The measurable turbidity ranges in absolute or Zeiss units are as follows:

Standard I–1	$0–6 \cdot 10^{-3}$
Standard I–3	$0–2 ; 10^{-2}$
Standard I–10	$0–6 \cdot 10^{-2}$
Standard I–30	$0–2 \cdot 10^{-1}$

Using a glass plate inserted into the measuring beam, the range of the individual standards can be increased by a factor of ten.

To convert the measured turbidity in per cent into absolute units, each instrument is equipped with a calibration factor C. The absolute, or Zeiss, units result from the quotient of the intensity of the scattered and of the incoming light.

The theoretical requirements for scattered light measurements are monochromatic light, no inherent absorption by the measuring liquid, and particles of the same average size.

The Askania turbidimeter operates as a single beam photometer. This apparatus determines the light transmitting capacity, or extinction, and absorption difference.

According to Lambert's law the relationship between light transmitting capacity and extinction is expressed by

$$I = I_0 \cdot 10^{-m.d}$$

$$\frac{I}{I_0} = 10^{-m.d}$$

$$m = \frac{1}{d} \cdot \log\left(\frac{I_0}{I}\right)$$

where I = intensity of measurement (measuring value); I_0 = intensity of extinction – 0 (transparency 100 %); m = extinction; d = path length in cm; and I/I_0 = light transmitting capacity.

The application of this relationship to the measurement of the suspended load results in the dependence of the extinction m on log D. The proportional relationship of the extinction to the suspended load results also in a logarithmic dependence on D for the concentration of suspended sediment.

The range can be adjusted to more or less intense degrees of turbidity by alteration of the path length d. Without any particular difficulties, the path length can be regulated at the measuring spot from 10 to 70 mm.

FACTORS INFLUENCING MEASUREMENT

It is generally known that the light transmitting capacity and light scatter depend not only on the concentration of the suspended material, but also on the size, colour, reflection and absorption capacity of the particles. Suspended sediments with insignificant variations in these characteristics allow turbidity measurement in mg solid matter per litre. According to previous experiments, hardly any waste or river water exists in which the turbidity remains constant in the properties in question for long periods. In such cases it is incorrect to carry out such a calibration, for this may well lead to data simulating a non-existent concentration of suspended sediments.

It is obvious that, in the case of turbidity measurements in coloured water, the coloration will cause considerable errors in absorption measurements. Turbidity and coloration have the same effect and thus confound any conclusion related to the individual components. In the scattered light measurements, however, the turbidity component only is taken into account. Colour does not produce any additional scattered light. As the red-sensitive photo-cell is generally used as a scattered light detector, the common yellow-brown colorations are of no importance. Experiments made with yellow solutions for scattered light measurements of one turbidity results in variations between the coloured and uncoloured liquids of about 2 per cent (relative). In the case of absorption measurements, the error was in the order of 30 per cent.

EXPERIMENTS MADE WITH THE SIGRIST PHOTOMETER

First of all, tests were carried out to determine the ranges for the individual turbidity standards. One the basis of samples of suspended sediments tested, which had been taken at the Rivers Obernach and Main, the ranges (approximate value) shown in Table 1 were obtained.

Table 1. *Range of the Sigrist photometer*

Turbidity standard	Suspended sediment	
	Obernach mg/l	Main mg/l
Standard I–3	0– 70	0– 30
Standard I–10	0– 200	0– 100
Standard I–30	0– 600	0– 300
Standard I–10 G	0–2 000	0–1 000
Standard I–30 G	0–6 000	0–3 000

The ranges are in the ratio 1 : 3. The differences between the two samples concerned are due to the particle size distribution (Fig. 1) and the composition. The suspended sediments of the River Main comprises up to 20 per cent organic components, whereas the suspended sediments of the Obernach contain mineral components only.

In the course of the calibration tests of the instruments, the linear dependence of the two corresponding measuring values are verified. Because of this linear dependence, the calibration curve can be computed by means of one single comparative measurement. The derivations of the measured values from the computer calibration curve are insignificant in relation to the errors of measurement.

Almost each sample of suspended sediment, as well as each particle size range of the same sample, results in a different calibration curve, as shown in Figures 2 and 3. Thus the ambiguity of the calibration is evident, especially since the granularity and composition of the suspended sediments may vary within the same water.

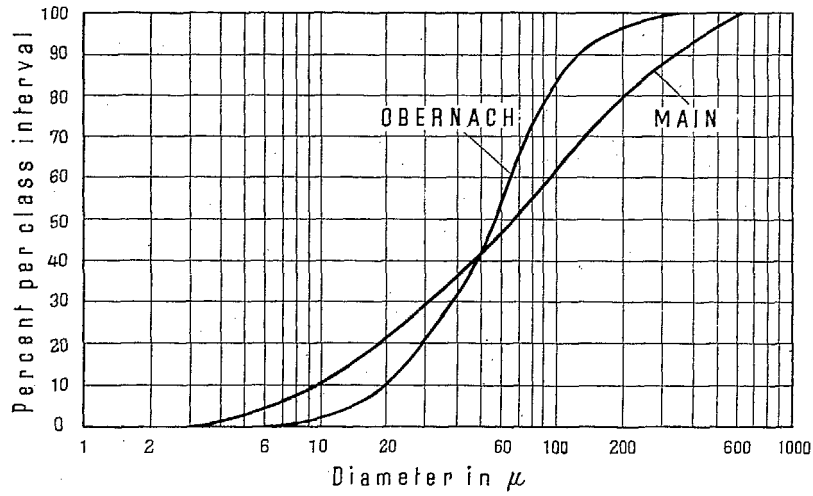


Figure 1 – *Cumulative curve of suspended sediments of the Rivers Obernach and Main*

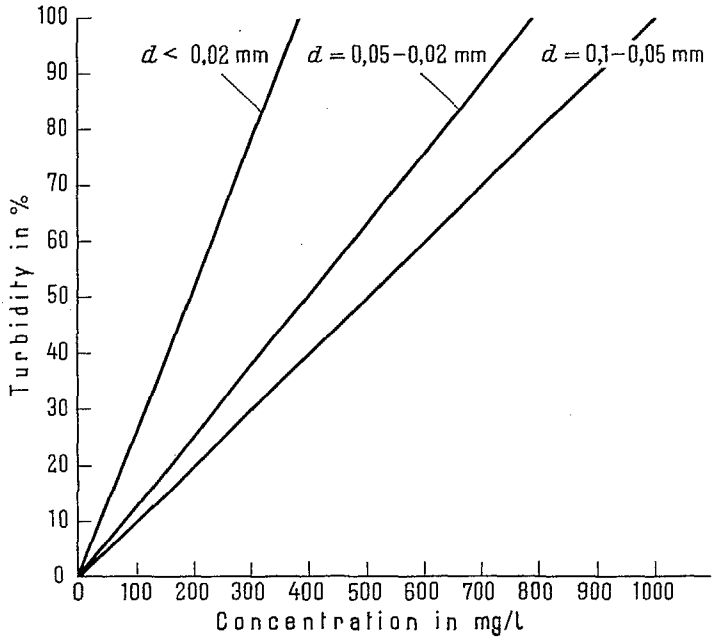


Figure 2 – *Calibration curves of the Sigrist photometer for different particle sizes*

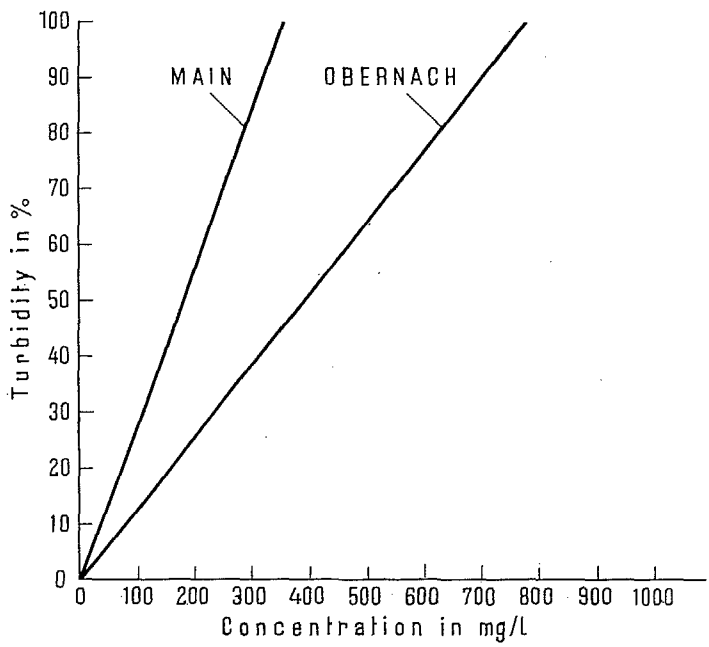


Figure 3 – *Calibration curves of the Sigrist photometer for suspended sediments of the Rivers Obernach and Main*

The Sigrist photometer was installed in the gauging station of the River Main at Markbreit. In spite of continuous supervision, defects both in the optical and electric system of the instrument resulted in breakdowns which could not be remedied quickly. Eventually the instrument had to be removed due to permanent breakdown of the recording mechanism.

As expected, the comparative measurements rendered different calibration curves, as shown in Figure 4. Thus, these tests had a completely negative outcome.

EXPERIMENTS MADE WITH THE ASKANIA TURBIDIMETER

The Askania turbidimeter has already been installed and tested at many gauging stations. The author's experiments are limited to a 1 year period on the River Isar near Munich. In Table 2, the ranges found by Brede (1962) are presented. The data refer to I_{min} = 10 per cent and I_{max} = 95 per cent light transmitting capacity.

Table 2. *Ranges of the Askania turbidimeter*

Path length (cm)	Extinction measuring range		Materials tested (cm^3/l)			
			kieselguhr		sewage	
	from	to	from	to	from	to
1	0.05	1.0	0.3	4.1	0.2	3.6
2	0.025	0.5	0.2	2.1	0.13	1.8
3	0.016	0.33	0.15	1.3	0.1	1.2
4	0.012	0.25	0.1	1.0	0.05	0.9
5	0.01	0.2	0.06	0.85	0.05	0.7
6	0.009	0.16	0.05	0.7	0.05	0.6
7	0.007	0.143	0.05	0.6	0.05	0.5

Assuming the density of the sediments to be $\rho = 0.5$ g/cm^3, the measuring range at $d = 1$ cm is 0–1800 mg/l. The comparative measurements at the River Isar rendered a range of 0–1000 mg/l. Thus the Askania turbidimeter has a considerably smaller range than the Sigrist photometer.

A calibration in cm^3/l or mg/l can be achieved under certain circumstances, but it is no easier to obtain than with the Sigrist photometer. The relationship of the two corresponding measured values is not linear, as is demonstrated by the calibration curves in Figures 5 and 6.

Figure 7 shows extinction as a function of the concentration, as computed from the calibration curve in Figure 6.

When operating, various defects which could not be mended at once, particularly when spare parts were needed, were inevitable. The instrument required continuous maintenance.

SUMMARY

According to the experiments so far undertaken, the photometric turbidity measurement for the determination of suspended load in running water has a limited field of operation. The calibration of a turbidimeter is generally very empirical and all the known problems of colorimetry also occur with photometric measurements.

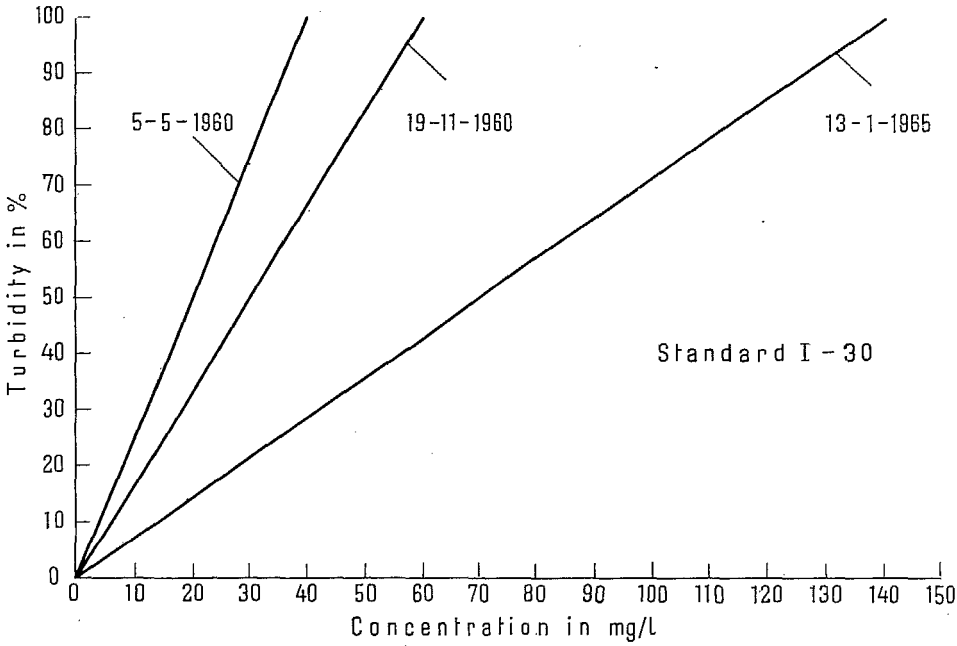


Figure 4 – *Calibration curves of the Sigrist photometer for individual comparative measurements at the Markbreit gauging station*

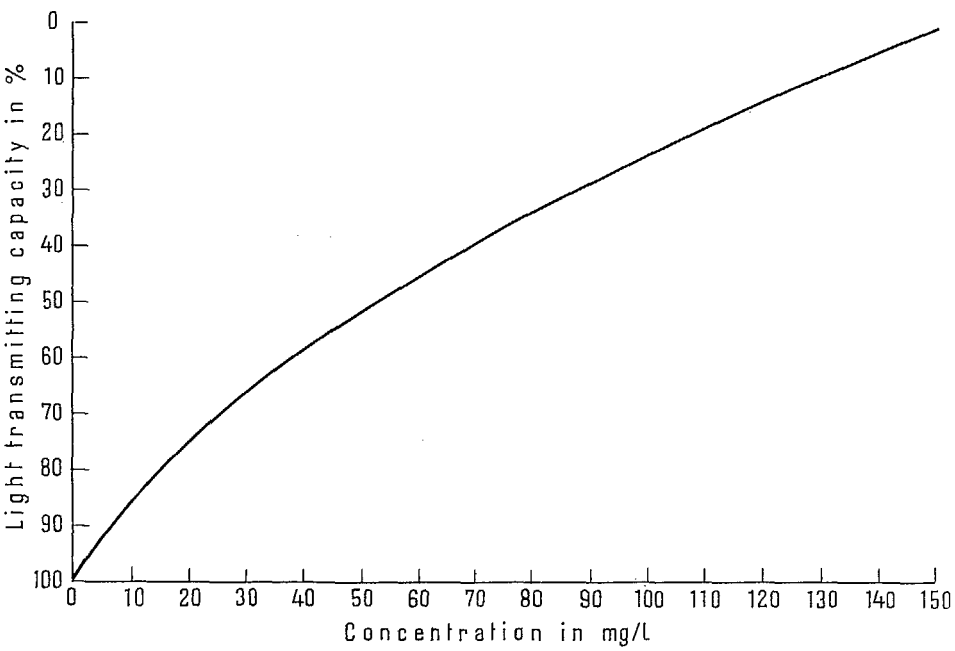


Figure 5 – *Calibration curve of the Askania turbidimeter at the Isar-Munich gauging station (path length, d = 7 cm)*

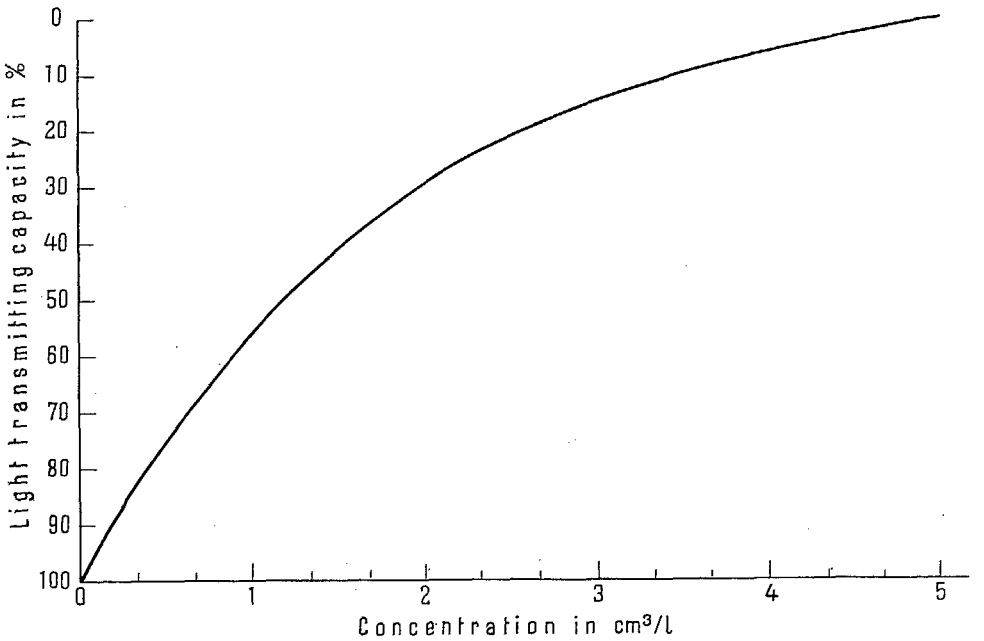


Figure 6 – *Calibration curve of the Askania turbidimeter for sewage according to Brede (path length, d = 7 cm)*

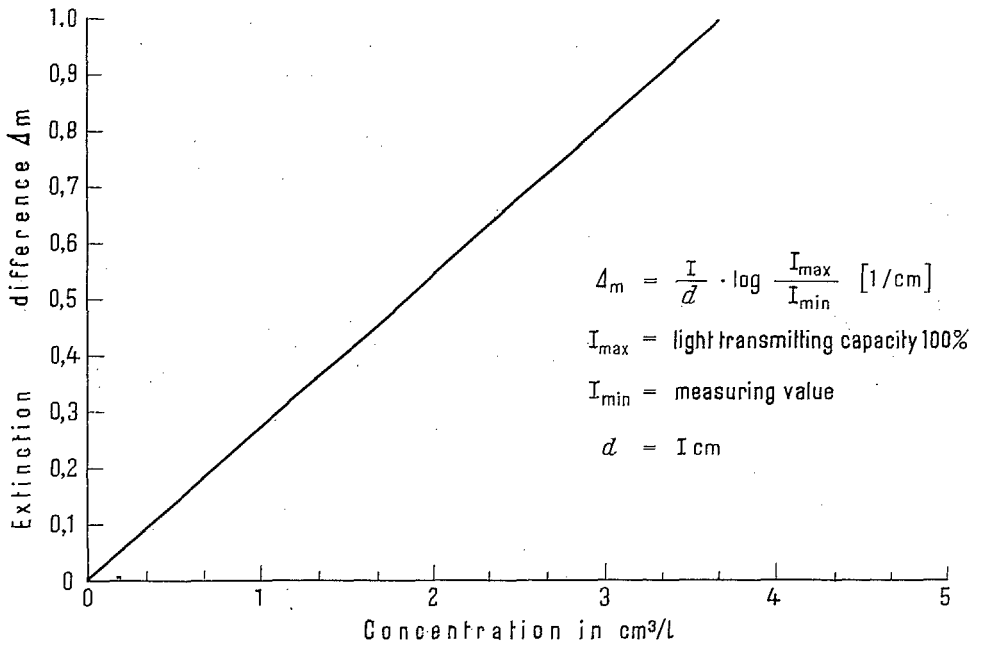


Figure 7 – *Curve of extinction as function of concentration, computed from the calibration curve in Fig. 6, according to Brede*

Considerable difficulties are met as to the installation and maintenance of the instruments; at the majority of gauging stations the instruments could not be installed due to lack of a current source.

A further disadvantage is the comparatively small range, which is far below the maximum concentration of suspended sediments in rivers. The advantage of continuous measurement of short-lived fluctuations of suspended load during floods must be mentioned; such a procedure is impossible with the conventional methods.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Brede, E. 1962. Die kontinuierliche Überwachung von Abwässern mit dem Askania-Trübungsmesser. *Askania-Warte*, vol. 60.

Gratz, E. 1966. Abwasserkontrolle mit Askania-Trübungsmesser in einer Brikettfabrik. *Askania-Warte*, vol. 68.

Husmann, W.; Krone, R. 1965. Erfahrungen mit dem Askania-Trübungsmesser bei der Abwasserkontrolle. *Askania-Warte*, vol. 66.

Joseph, H. 1950. Durchsichtigkeitsregistrierungen als ozeanographische Untersuchungsmethode. *Deutsche Hydrographische Zeitschrift*, vol. 3, no. 1/2.

Klein, H. 1952. Schwebstoffmessung mit dem lichtelektrischen Trübungsmesser. Bundesanstalt für Gewässerkunde, Mitteilung no. 38, Koblenz.

Lüneburg, H. 1950/51. Über Messung und Bedeutung der Sinkstoffe in Elbe–und Wesermündung. In *Wasser, Jahrbuch der Wasserchemie und Wasserreinigungstechnik*, vol. XVIII.

Malz, F. 1962. Möglichkeiten der kontinuierlichen Überwachung des Abwassers und der Gewässer (Teil II). *Wasser, Luft und Betrieb*, vol. 5.

Subcommittee on Sedimentation. 1963. A study of methods used in measurement and analysis of sediment loads in streams. Report no. 14. Washington, D.C.

Teufert, O. 1964. Abwässerkontrolle mit dem Askania-Trübungsmesser. *Askania-Warte*, vol. 64.

Veit, R. 1961. Der Trübungsmesser für Abwasser als kontinuierlich arbeitendes Betriebsmessgerat. *Askania-Warte*, vol. 57.

Weiler, F. 1958. Ein robuster und vielseitiger Zweistrahlphotometer für Laboratoriums– und halbindustrielle Verwendung. *Chemische Rundschau*, no. 18.

Weiss, B. 1959. Kontinuierliche Prozesskontrolle durch optische Messungen. *Neue Zürcher Zeitung*, vol. 4, February 1959.

DISCUSSION

Kohler: I should like to know whether Dr. Burz has simultaneously measured the temperature and whether he has looked for relationships between changes of the temperature of the water and an increase of suspended load transport. We have found in Alpine rivers that the temperature increased approximately one or two hours before an increase of discharge and–in connexion with it–of sediment transport.

Burz: These relationships have been reported repeatedly, but I have some doubts. Certainly there is some relationship since the temperature influences the kinetic and dynamic viscosity and the dynamic viscosity influences the velocity of the free fall of the grains. But I cannot believe in a major influence of temperature since other factors in a river are more important. There are no rules for the time intervals and for the appropriate moment for making the measurement. According to our experience, measurements should be made hourly during a flood, but I admit that this has been only done so far in exceptional cases. In particular, the hours during the night have been left out. Therefore we do not yet have a precise picture of the suspended load

transport throughout a flood period. I should like to repeat that no rule for measurements exists and that the only alternative is to measure as often as possible.

Mundt: When investigating the concentration of suspended load in the inlet of a power station, we found enormous variations and, therefore, regard one point as insufficient for photometric measurements.

Burz: The most difficult problem is, in fact, the selection of a representative station for measurements since an integrating or even a full measurement of the whole cross-section is only possible in exceptional cases. Usually only a single measurement can be carried out, sometimes in one or two verticals. The solution of this problem is nearly impossible since one point will never be representative.

Milisic: Did you take into account the effects caused by microplants and micro-biological organisms?

Burz: The influence of micro and macro-organisms is evident. In the case described by me, I worked with water with a negligible organic content. I remember, however, one case at the River Main where plants repeatedly blocked the device. In water with a turbidity caused by organic matter, any calibration is very problematic, or may even be impossible.

Grindley: The determination of concentration of suspended solids by classical gravimetric methods is time consuming and, therefore, costly. We have to use indirect methods of analysis in order to reduce the cost of obtaining a large number of results. If *in situ* measurements are to be made by other methods, one important difficulty is the coagulation of the suspended particles. In the estuaries of the River Mersey and the River Thames, the flocs may have a diameter of up to 2 mm. The optical density of such suspensions is different from that of the same material, but with the flocs dispersed.

If one accepts the disadvantages of the optical method, it is possible to obtain certain types of information which are not readily or cheaply obtained by any other technique. For example, in the Humber estuary a recording siltometer was in operation for eighteen months and it was then possible to relate concentration of solids to the amplitude of the tide, the phase of the tide cycle and to the flow of fresh water into the estuary. This enabled a programme of pumping dust into the docks to be arranged so that the minimum amount of solids was transferred to the docks.

It is necessary to calibrate turbidimeters almost continuously. During about ten years' work on the estuary of the River Thames, samples were examined in the laboratory by optical methods, but 25 per cent of the samples were analysed gravimetrically. For a number of years we have had a siltometer in the United Kingdom which overcomes most of the difficulties associated with the optical method. It measures the ratio between the scattered and reflected light when a beam of light is passed through the suspension. The background colour of the water is not important.

Burz: I am convinced that the photometric method will be further developed and that it has advantages in comparison with the gravimetric method. At present the existing instruments are not adequate for the quantitative determinations that are

needed. Qualitative indications of the turbidity are rather useless. The present equipment requires permanent maintenance and, therefore, only has a limited use for practical purposes in hydrological services. Their number is also insufficient; we face also a certain problem with spare parts, which can be replaced only by a specialist.

Hinrich: 1. The Water and Navigation Authority, Hamburg, has installed a measuring device for turbidity, the Sigrist system, in the measuring ship *Kulgelbake.* Tests have been made with normal water from the Elbe and with the same water coloured with potassium dichromate. The comparison of turbidity did not reveal any differences, though the second probe had been coloured very intensively.

2. The measurements of turbidity may be regarded as a valuable addition to gravimetric measurements, particularly for periods between gravimetric measurements, e.g. during a flood or for the determination of the influence of ships on the turbidity of water. We have found that during night hours, when navigation ceases, the suspended load lessens.

3. I should like to add that the Water and Navigation Authority has installed about ten measuring chambers in pillars of weirs in some of its navigable waterways. Usually three inlets are foreseen, one of them 1 m under the headwater level, one of them 1 m above the bottom, and the third one in between the two others.

4. The Bavarian Hydrological Services usually take two gravimetric probes (10 *l*) per week; the Federal Institute for Hydrology, however, takes five to seven probes per week, generally from Monday to Friday, and also during the weekend when personnel are available. During floods additional probes are taken. In rivers with weirs, probes are normally taken in the tail water, taking into account that suspended load is adequately distributed by the screws of the ship.

Gruber: Although the measurement of turbidity has many possible errors, one basic error should be mentioned. This is the definition of the suspended load. The usual definition is that suspended load is that matter which is kept suspended by the turbulence and flow. Certain grains, therefore, belong to the suspended load during high velocities and will belong to the bedload during low velocities.

Burz: You have mentioned an additional subject—the limits between bedload and suspended load. You are right that the limit does not depend on the size of the grain. However, usually the amount of grains > 0.2 mm, is negligible in the suspended load, but this is only a rough indication. Normally special tests are necessary.

Rodier: En ce qui concerne la continuité des mesures de turbidité, le problème varie suivant qu'il s'agit de grandes ou de petites rivières. Nous avons eu la possibilité de faire en Afrique des études méthodologiques sur cette question pour des surfaces variant de quelques hectares à 600 000 km^2. Sur les grands cours d'eau drainant plus de 30 000 à 50 000 km^2 en pays pas trop accidenté, la mesure continue ne nous semble pas nécessaire. Nous faisons un prélèvement ou deux chaque jour en un ou deux points soigneusement choisis dans la section. De temps en temps, une mesure effectuée sur toute la section (en 10 points par exemple) permet d'établir la corrélation entre la turbidité en un point et la turbidité moyenne dans la section. Cette méthode est également utilisée en URSS. Par contre, sur des cours d'eau drainant quelques hectares, il faudrait un prélèvement toutes les deux ou toutes les cinq minutes pendant la crue. Pour les petits et les moyens cours d'eau, nous sommes bien d'accord avec l'orateur : il faut absolument trouver un moyen de mesure ou de repérage continu.

Burz: Your information on major rivers is appreciated. I should be interested in data and additional information about big rivers. I agree with Mr. Rodier that in these cases even during flood periods one daily measurement would be quite sufficient, but up until now we have had no information on the suspended load transport in the mouths of big rivers, such as the Ganges or Congo. It is obvious that the variations in the sediment transport increase the smaller the river.

Sauzay: De nouvelles méthodes, utilisant les traceurs radio-actifs ou la radio-activité naturelle des sédiments, sont présentées dans une autre communication. Elles ont été utilisées avec succès dans deux estuaires français : ceux de la Loire et de la Gironde.

A method for measuring the quantity of bedload transported by short flood waves

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Abstract. The transport of bedload in rivers draining small basins is carried out by flood waves. The determination of the quantity of bedload transported during the passage of a flood wave is not possible by use of classical bedload samplers because of the high velocities involved. For such cases a new method is proposed for the measurement of bedload. Special sediment traps are installed at different positions in a cross-section. The size and form of the sediment traps were determined by experiments in the laboratory and in the field. The traps are designed to retain all material caught in them. The method provides the possibility of determining the mean bedload discharge, its distribution in the cross-section and the total mass of bedload transported by the flood wave.

MÉTHODE DE MESURE DES TRANSPORTS DE FOND ENTRAINÉS PAR DE COURTES ONDES DE CRUE

Résumé. Le transport des matériaux de fond dans des rivières qui ont un petit bassin et un caractère torrentiel se fait exclusivement lors du passage des ondes de crue. Les vitesses élevées qu'on rencontre lors de ces phénomènes ne permettent pas d'utiliser les appareils classiques de prélèvement pour déterminer le débit de charriage des matériaux de fond transporté par le courant. Dans ce cas, il convient donc d'appliquer une méthode nouvelle, telle que la suivante. Des caisses spéciales de prise de sédiments sont installées en différents endroits sur une section transversale de la rivière : leur forme et leurs dimensions sont déterminées par des tests en laboratoire et *in situ*. Les matériaux de fond qui ont atteint le bord amont de ces caisses sont retenus par elles. Cette méthode permet de déterminer le débit moyen des matériaux de fond lors d'une onde de crue, sa distribution dans une section transversale de la rivière et le volume total des matériaux de fond transportés.

METODO DE MEDICIÓN DE ARRASTRE DE FONDO EN ONDAS CORTAS DE CRECIDA

Resumen. El arrastre de fondo en ríos con pequeñas cuencas y carácter torrencial se debe casi exclusivamente al paso de ondas de crecida. La determinación del arrastre de fondo transportado durante el periodo de la onda de crecida no es posible a través de los clásicos tomadores de muestras, debido a las velocidades elevadas.

Para este caso se propone un nuevo método para la medición del arrastre de fondo en forma rápida. Se instalan unas cajas de captura especiales en diferentes posiciones en la sección transversal del río. El tamaño y la forma de las cajas de captura se determina experimentalmente en laboratorio y en condiciones naturales. El material del arrastre de fondo que alcanza el borde frontal de la caja de captura se mantiene dentro de ella, dando el método la posibilidad de determinar el arrastre medio de fondo, su distribución en una sección transversal y el arrastre total de fondo transportado durante la onda de crecida.

МЕТОД ИЗМЕРЕНИЯ ДОННЫХ НАНОСОВ, ПЕРЕНОСИМЫХ В ПЕРИОД КРАТКОСРОЧНЫХ ПАВОДКОВ

Резюме. Перенос донных наносов в реках, дренирующих малые бассейны, осуществляется перемещением паводочной волны. Определение расхода донных наносов, переносимых потоком в течение паводочной волны, с

помощью классических батометров для донных наносов оказывается невозможным из-за высоких скоростей. В таких случаях предлагается новый метод для суммарного измерения донных наносов.

Специальные ящики-уловители устанавливаются в различных местах в поперечном сечении. Размер и форма ящиков-уловителей устанавливается экспериментально в лаборатории и полевых условиях. Донные наносы, достигнув переднего края ящика, удерживаются в нем. Метод дает возможность определить средний расход донных наносов, его распределение в поперечном сечении и общее количество донного материала, переносимого паводочной волной.

INTRODUCTION

The sediment discharge of rivers can be determined from measurements of suspended load, bedload and the mineral substances dissolved in the water. There are numerous methods for the determination of suspended load and dissolved substances. On the other hand, the methods for measuring bedload are few and of low accuracy. The main reason for this is the difficulty of working in the boundary layer on the river bed. There are also the problems caused by the presence of the instruments on the river bed.

Various bedload samplers have been tried in the past and have been found unsuitable. Even the samplers in current use in many countries are not perfect. Although the principle of the bedload sampler is very simple, the construction of a sampler required to give precise measurement is very complicated.

The main disadvantages of the bed-load samplers are:

1. The difficulties of adapting them to the form of the rough bottom of the river, thus allowing some sediment to be stored underneath the instrument.
2. The instruments must be sufficiently heavy not to be carried away by the current, but this can lead to scooping of the sediments.
3. The difficulties of controlling the instrument's position, particularly in muddy waters and at considerable depths.
4. There is the contradiction between the need for not keeping the instrument on the bottom for too long a time so as to escape the harmful effects of erosion and the need to maintain its position. The non-uniform movement of the bedload must be taken into account.

Measurement of bedload during the passage of floodwaves, when the most rapid movement of bedload is taking place, is not always possible because of the high velocities experienced. For these cases some other methods need to be found. The strip method developed for the determination of bedload transport for rapid-moving short flood waves produced interesting data about the process of bedload transport and some quantitative relationships between the transport and its formative factors.

MEASUREMENT OF BEDLOAD DISCHARGE

Special equipment installed in the river bed is used for the strip method of measuring bedload. It consists of parallelepipeds of sizes depending upon the character of the flood waves and the sediment in the river. Their widths are usually fixed at 1 m, so that the bedload measured can be referred to a unit width of the river bed. In our experiments, the other dimensions varied between 0.5 and 1.5 m for the depth and 1.0 and 3.5 m for the length of the apparatus. The sediment traps are installed in the

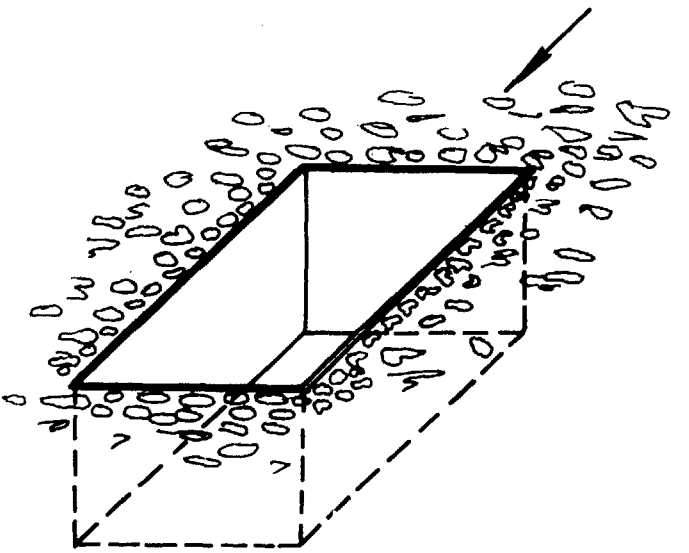


Figure 1 – *Diagram of a trap in the river-bed*

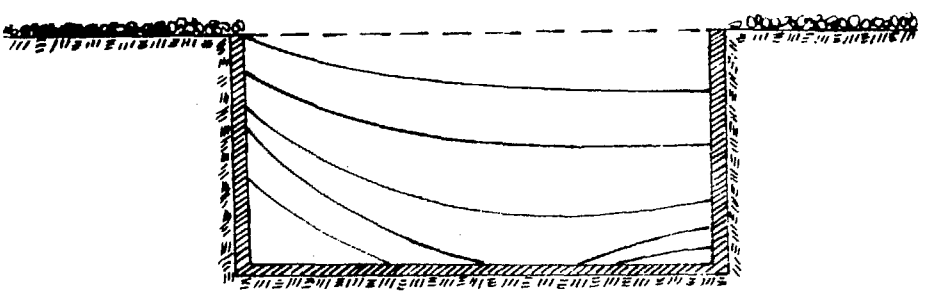


Figure 2 – *Process of filling the trap*

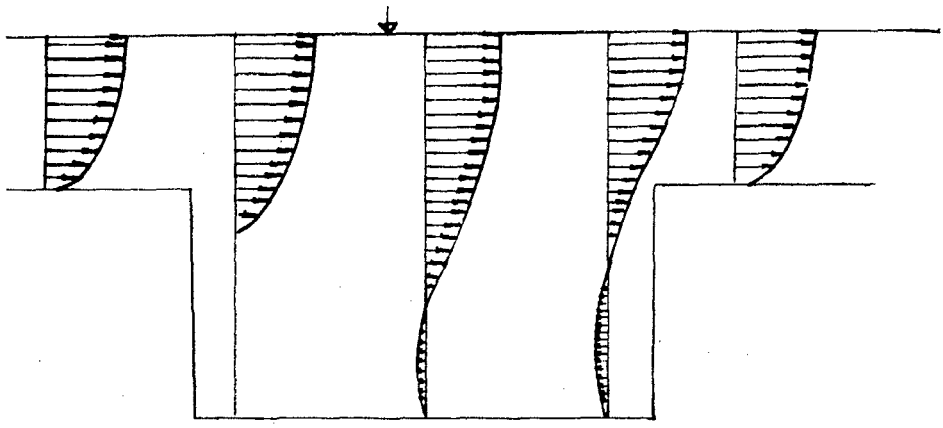


Figure 3 – *Velocity-structure of the flow in and above the trap*

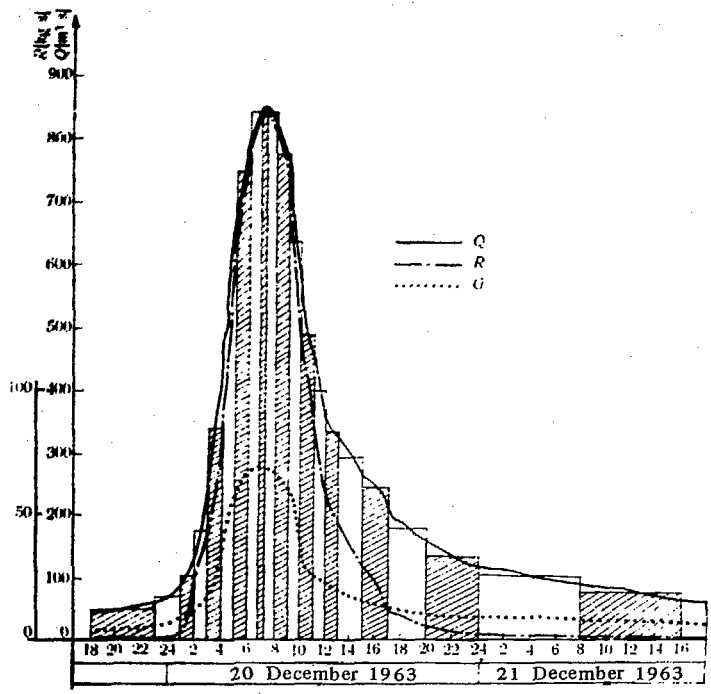


Figure 4 – *Stream hydrograph presented as a histogram*

bed sediments with their upper edges coinciding with the surface of the river bed (Fig. 1). The sediment traps collect the bedload reaching their front edges. During the passage of a flood wave and from the mass collected, the average quantity transported is determined.

The number of sediment traps used depends upon the width of the river. One sediment trap is sufficient in rivers less than 10 m in width. In wider rivers, the number of sediment traps is fixed by the requirement for the minimum distance between sediment traps to be 10 to 15 m. The sediment traps must be positioned in the cross-section so that the front side faces the direction of flow. The most suitable places are at representative verticals in the cross-section where the velocity and sediment are usually measured.

The rivers must have a fluctuating regime with rapidly passing flood waves in order to install the sediment traps and to avoid the difficulties connected with their use and cleaning. The measurement of bedload in the sediment trap is carried out as follows: the distance between the upper edge of the sediment trap and the surface of the bedload in the sediment trap is measured by means of a graduated staff or a current meter rod. Places for measurement are selected in order to sample the size and configuration of the bedload in the best possible way. The volume of the bedload can be determined as a result of elementary calculations from a knowledge of the total volume of the sediment trap. Some measurements of bedload density must be made to transform the volumes into weights.

Sediment traps were installed in four rivers having drainage basins from 6 to 500 km^2 and bed-widths from 5 to 80 m.

Various investigations were carried out to clarify the action and efficiency of the sediment traps. From the hydraulic point of view, the sediment trap can be considered to be a sudden obstruction, but its limited size does not allow much of a deformation of the flow structure. The water in the sediment trap is almost stationary or slowly circulating around a horizontal axis crossing the flow direction. This contributes to the detention of the particles falling into the sediment trap. This process of filling continues until about 70–80 per cent of the total volume of the sediment trap is full. Above this value the smaller bedload particles are not retained. The process of filling the sediment trap is shown in Figure 2. At the beginning, the material is accumulated in the front part of the sediment trap, and only some of the small-size particles reach the rear. An examination of the velocity-structure of the flow in the sediment trap and above it (Fig. 3) shows that considerable changes are not observed. The maximum velocities of the eddy formed in the sediment trap reach 30–35 per cent of the value of the velocities of the river above it. These velocities cannot carry particles up from the sediment trap through the transit zone.

The filling of several sediment traps installed at one of the measurement stations during various flood waves, as well as some other characteristics of the flood wave are shown in Table 1. A great variation in the amount of filling is evident, and this is also an indicator of the distribution of bedload along the width of the river.

Table 1. *Data from the filling of several sediment traps*

Maximum water stage (cm)	Maximum discharge (m^3/sec)	Maximum sediment discharge (kg/sec)	Volume of bedload detained in the sediment trap			
			No. 1 (m^3)	No. 2 (m^3)	No. 3 (m^3)	No. 4 (m^3)
147	179	1 180	0.48	0.52	1.29	1.92
197	235	940	0.46	0.57	2.16	2.73
178	187	442	1.16	2.22	3.94	4.05
144	122	278	0.12	0.60	2.37	3.12
153	138	290	0.24	0.68	2.31	2.96
220	295	1 000	0.22	0.62	1.32	1.95
160	165	419	0.38	0.48	1.26	2.03
197	262	1 460	1.94	1.92	2.87	3.43
170	224	1 500	1.98	2.06	3.50	3.83
270	450	2 730	1.78	1.84	3.20	3.40

CALCULATION OF THE QUANTITY OF TRANSPORTED BEDLOAD

Bedload volumes in the sediment trap multiplied by their densities give the mass or the total bedload discharge that has passed through 1 m width of cross-section of the river for a particular time interval. To find the relationship between the quantity of bedload transported and the hydraulic characteristics of flow, it is necessary to process the different data obtained during the passage of a flood wave. This relationship is very useful in calculating the bedload discharge at a gauging station from the water levels or discharges recorded there. To this end it is necessary to make concurrent observations of the velocity distributions at different water levels, the variation of the water surface gradient, and the sediment discharge and particle size distributions. The stream hydrograph is divided into different time intervals and presented as a histogram, flow characteristics being constant for each interval (Fig. 4). Similarly the cross-section of the river is also divided into zones depending on the number of the sediment traps. In these zones the flow characteristics during each time interval are constant and correspond to those over the sediment traps. Using all the data from the observations and calculations and the approximate space-time model of flood wave movement through the gauging station, some calculations for the bedload discharge were made from several formulae given by different authors. The total mass of bedload that has passed through the station during the flood wave, W_s can be calculated fom the expression

$$W_s = \sum_{j=1}^{m} W_{cj} \cdot b_j .$$

On the other hand this mass must correspond to the relationship:

$$W_s = \sum_{i=1}^{n} \sum_{j=1}^{m} G_{ij} \cdot t_i \cdot b_j$$

where W_{cj} = the mass of bedload in the sediment trap; b_j = the width of strip of the river cross-section corresponding to that of sediment trap; t_i = the time interval

of a single bar of the flood wave hydrograph; G_{ij} = the bedload discharge determined by means of formulae.

The results obtained from these calculations were compared with the data from the sediment trap. This comparison allowed the most suitable formula to be selected for every cross-section as well as a simplified relationship $G = f(Q)$.

CONCLUSION

The strip method for the measurement of bedload passing through a gauging station has a number of advantages compared with the classical bedload samplers when used for short duration flood waves. Sediment traps of a correct size can detain all the bedload reaching the front edge. In the case of longer sediment traps, some bedload gets in through the side edges causing an increase in the volume of the bedload compared with the real value. This error cannot be determined precisely at the moment.

If the sediment trap is not well constructed, it is possible for it to be completely filled in some cases, and this does not allow the exact determination of the mass of bedload. Data received from such a sediment trap can only be used as an approximate measure.

The use of sediment traps allows the difficult work of taking bedload measurements during flood wave to be avoided.

Statistical parameters of distribution of granulation indicating suspended sediment and bed sediment

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Abstract. Grain diameters based on insufficient statistics are often used to characterize the distribution of granulation of total sediment. Based on the typical form of the distribution of granulation, statistical parameters are presented, and their values for typical granulation mixtures are given. The distribution of grain number and grain surface can be derived from the usual distribution of granulation. The degree of fineness and the degree of uniformity are derived from Kneschke and Schoch's formula.

PARAMETRES STATISTIQUES DE DISTRIBUTION DE LA GRANULOMETRIE UTILISES COMME INDICES DES TRANSPORTS EN SUSPENSION ET DES TRANSPORTS DE FOND

Résumé. Pour mettre en évidence la répartition de la granulométrie des sédiments on utilise souvent des diamètres de grain qui ne sont pas suffisamment justifiés statistiquement. En se basant sur la forme caractéristique de la courbe de répartition de la granulométrie, l'auteur propose des paramètres dont les grandeurs sont fonction des répartitions granulométriques types. La répartition du nombre des grains et de leur superficie peut être déduite de la courbe de répartition granulométrique usuelle. Par extension, une publication de Kneschke et Schoch donne le degré de finesse et de régularité des matières en suspension et des galets.

PARÁMETROS ESTADÍSTICOS DE DISTRIBUCIÓN DE GRANULOMETRÍA QUE INDICAN EL SEDIMENTO EN SUSPENSIÓN Y EL ARRASTRE DE FONDO

Resumen. Para caracterizar la distribución de granulometría del sedimento total se utilizan con frecuencia diámentros de grano sin el suficiente fundamento estadístico. Con base a la forma típica de la distribución de la granulometría se presantan parámetros definidos estadísticamente, y se dan sus valores para mezclas típicas. A partir de la distribuciones de número de grano y de superficie de grano. A partir de la fórmula de Kneschke y Schoch, se obtiene el grado de finura y el de uniformidad.

СТАТИСТИЧЕСКИЕ ПАРАМЕТРЫ РАСПРЕДЕЛНИЯ ГРАНУЛОМЕТРИЧЕСКОГО СОСТАВА ВЗВЕШЕННЫХ И ДОННЫХ НАНОСОВ

Резюме. Диаметры частиц, статистически недостаточно обоснованные, часто используют для характеристики распределения гранулометрического состава наносов. На основании типичной формы распределения гранулометрического состава представлены статистические параметры и даны их значения для типичных смесей. Распределение числа частиц и поверхности частиц могут быть установлены по обычному распределению гранулометрического состава. Степень измельченности частиц и степень их однородности установлены по формуле Кнешке и Шоха.

To characterize suspended sediment and bed sediment, grain diameters taken from the distribution of granulation are generally used, together with parameters derived from them. This means that widely differing grain diameters are used, thus making a comparison of results difficult. A unique system of parameters that is statistically determined and which describes the whole range of the distribution of granulation is, therefore, desirable.

STATISTICAL PARAMETERS OF THE DISTRIBUTION OF GRANULATION

The distribution of granulation is most frequently S-shaped and can, therefore, be described by the log-normal distribution (Kezdi, 1964) ; hence, it can be interpreted as a probability distribution. Figure 1 shows three distribution curves which strictly obey the log-normal law. If this type of distribution is adopted, its parameters d and s_g offer themselves as parameters of the distribution of granulation as mean value and dispersion respectively. To measure these reliably, granulation must be determined by screen analysis with a sufficient number of grain fractions.

From the distribution of granulation, statistically sensible grain diameters $l(d_{50} + k\sigma)$ can be obtained, consequently the median value

$$d_0 = d(50\ \%) = d_{\mathrm{Med}} \tag{1}$$

further

$$\begin{aligned} d_1 &= d(-\sigma) &&= d(15.87\ \%) \\ d_2 &= d(+\sigma) &&= d(84.13\ \%) \\ d_3 &= d(-2\sigma) &&= d(\ 2.28\ \%) \\ d_4 &= d(+2\sigma) &&= d(97.72\ \%) \\ &\ldots\ldots\ldots\ldots\ldots\ldots \end{aligned} \tag{2}$$

From these grain diameters result:
the degrees of inequality

$$\begin{aligned} U_1 &= d_0/d_1 = U(-\sigma) \\ U_2 &= d_0/d_2 = U(+\sigma) \\ U_3 &= d_0/d_3 = U(-2\sigma) \\ U_4 &= d_0/d_4 = U(+2\sigma) \end{aligned} \tag{3}$$

the degree of graduation

$$\begin{aligned} C_{12} &= \frac{d_0^2}{d_1 \cdot d_2} = C(\sigma) \\ C_{34} &= \frac{d_0^2}{d_3 \cdot d_4} = C(2\sigma) \end{aligned} \tag{4}$$

and the ranges

$$\begin{aligned} R_{12} &= d_2 - d_1 = R(\sigma) \\ R_{34} &= d_4 - d_3 = R(2\sigma) \end{aligned} \tag{5}$$

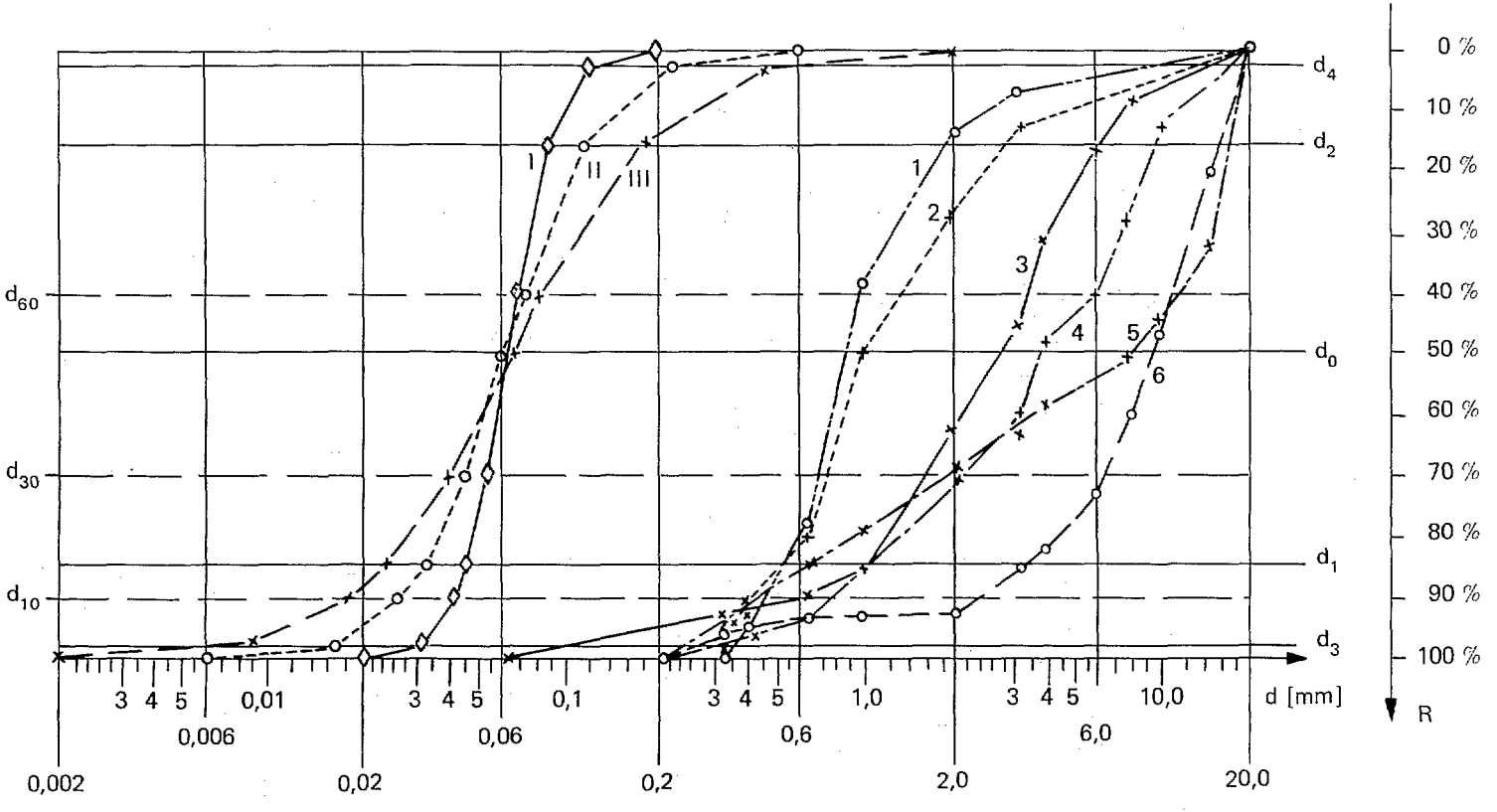


Figure 1 – *Distribution of granulation (parameter values given in Table 1)*

Table 1. *Parameters of distributions of granulation in Figure 1*

	I	II	III	1	2	3	4	5	6
d_w [mm]	0.0602	0.0488	0.0434	0.842	1.03	1.07	1.93	1.69	2.93
$U = d_{60}/d_{10}$	1.47	3.28	4.48	2.13	3.32	5.66	7.82	27.2	4.91
$C = d_{30}^2/d_{60}d_{10}$	0.954	1.19	0.986	1.08	0.983	1.14	0.875	0.707	1.72
d_0 [mm]	0.0625	0.0600	0.0650	0.860	1.00	2.75	3.75	7.80	8.40
$U_1 = d_0/d_1$	1.39	1.95	2.73	1.65	1.92	2.72	3.64	12.4	2.62
$U_2 = d_0/d_2$	0.722	0.521	0.374	0.475	0.360	0.451	0.407	0.459	0.542
$U_3 = d_0/d_3$	1.92	3.68	7.18	2.46	3.12	25.4	11.7	3.18	3.21
$U_4 = d_0/d_4$	0.525	0.274	0.145	0.0905	0.0736	0.186	0.223	0.407	0.454
$C_{12} = d_0^2/d_1d_2$	1.003	1.017	1.021	0.785	0.692	1.23	1.48	5.68	1.42
$C_{34} = d_0^2/d_3d_4$	1.007	1.009	1.042	0.223	0.229	4.73	0.262	12.9	14.6
$R_{12} = d_2 - d_1$	0.0415	0.0842	0.150	1.29	2.26	5.09	8.17	16.4	12.3
$R_{34} = d_4 - d_3$	0.0864	0.203	0.358	9.15	13.3	14.7	16.5	19.0	18.2
$F_s = 1/\bar{d}$	25.20	40.75	151.8	0.49	0.67	7.56	0.79	1.07	1.14
$F_G = 1/\bar{d^3}$	2.27	1.11	1.19	2.08	2.06	4.05	2.07	2.04	1.98
	10^4	10^6	10^8	10^1	10^1	10^3	10^1	10^1	10^1
$G_S = 1/\sigma_F$	23.05	35.63	47.90	0.406	0.374	0.826	0.297	0.604	0.815
$G_G = 1/\sigma_N$	0.217	0.0471	0.0162	0.127	0.155	0.198	0.195	0.278	0.829

Table 1 gives these parameters for the distribution curves in Figure 1. Distributions I to III are exact log-normal distributions and are similar to artificial distributions of granulation with given parameters (Hansel, 1968a). Distributions 1 to 6 are distributions of granulations from samples of bed material taken from below a flood storage reservoir in flat territory in 1967 (see Hansel, 1968b). For the usual analysis of the distribution of granulation it is proposed to use d_0, U_1, U_2, C_{12}, R_{12}; in addition, in the case of a more detailed analysis of grain size than is customary, C_{34} and R_{34} are used.

Further parameters of k (with $k \geqslant 3$, integer) are appropriate only for special investigations.

U_1 is very strongly correlated with the conventional degree of inequality $U = d_{60}/d_{10}$ and so it can be substituted for U; based on the distributions I to III and 1 to 6 it gives the linear equation $U_1 = 0.63 + 0.42\,U$. With the proposed parameters all ranges of the distribution of granulation can be described more reliably than with the parameters U and C used previously. This is of particular relevance for filter design, channel stability and so on.

FURTHER STATISTICAL PARAMETERS OF THE GRAIN SIZE DISTRIBUTION

The grain size distribution is characterized not only by the distribution of granulation on the basis of weight (or, for equal density of grain fractions, as the basis of volume), but also by the distribution functions of surface and grain number. Kneschke and Schoch (1959) have shown that, for grain mixtures, their distribution of surface is of special importance (for instance, products of mills, dusts) in order to derive from the distribution of surface

$$f(d) = \frac{V}{\lambda \,.\, F} \,.\, \frac{R'(d)}{d} \tag{6}$$

a degree of fineness

$$F_s = 1/\overline{d}$$

$$= \frac{\lambda \,.\, F}{V} = \frac{1}{a} - \int_a^b \frac{R(d)}{d^2}\, \mathrm{d}d \tag{7}$$

where d = grain diameter, in the range $a \leqslant d \leqslant b$ the Heywood factor ($\lambda_{\text{sphere}} = 1/6$); F = the total surface of grain mixture; V = the total volume of grain mixture; and $R(d)$ = the residue function of grain mixture.

As degree of uniformity is derived in (4)

$$G_s = 1/\sigma_F$$

$$= \frac{1}{\overline{d}} \cdot \frac{1}{\sqrt{\frac{1}{\overline{d}} \cdot \int_a^b R(d) \,.\, \mathrm{d}d + \frac{a}{\overline{d}} - 1}} \tag{8}$$

$$= \frac{F_s}{\sqrt{F_s \left(\int_a^b R(d) \,.\, \mathrm{d}d + a \right) - 1}} \,.$$

The degrees of fineness and of uniformity following Kneschke and Schoch (1959) are suitable for characterizing samples of suspended material because the surface of grain mixture is the main parameter, as for instance is evident by the importance of the Reynolds number in the settling velocity.

For bed sediment the surface is not the main parameter, as the importance of the Froude number in relation to movement of bed sediment indicates. For samples of bed sediment it is necessary to assume a distribution function describing the distribution of volume. For that purpose the distribution of grain number is suitable and it can be shown that a derivation analogous to (4) gives rational results.

Starting from the distribution function of grain number

$$n(d) = -\frac{V}{\nu \,.\, N} \cdot \frac{R'(d)}{d^3} \tag{9}$$

one gets for the degree of fineness for samples of bed sediment

$$F_G = 1/\overline{d}^3$$

$$= \frac{V}{\nu \cdot N} = \frac{1}{a^3} - \int_a^b \frac{R(d)}{d^4}\,\mathrm{d}d = \frac{1}{\overline{V}} \tag{10}$$

where ν = a proportionality factor ($\nu_{\text{sphere}} = \pi/6$); and N = the total grain number of the grain mixture.

The degree of uniformity for samples of bed sediment is then defined by

$$G_G = \frac{1}{\sigma_N} = \frac{1}{\overline{d}} \cdot \frac{1}{\sqrt{\frac{1}{\overline{d}}\left(4 \cdot \int_a^b \frac{R(d)}{d^3}\,\mathrm{d}d - \frac{2}{a^2} + \frac{1}{\overline{d}^2}\right) + \frac{1}{\overline{d}^3}}}$$

$$= \frac{F_s}{\sqrt{F_s\left(4 \cdot \int_a^b \frac{R(d)}{d^3}\,\mathrm{d}d - \frac{2}{a^2} + F_s^2\right) + F_G}} \cdot \tag{11}$$

For practical calculations it is appropriate to determine the integrals by differential calculus. The values a and b can be derived from the graphical representation of $R(d)$. Eventually they can be calculated as

$$a = d\,(-3\sigma) \quad \text{and} \quad b = d\,(+3\sigma).$$

Because the relationships yield:

$$V(d) = \alpha \cdot \frac{\pi}{6} \cdot d^3 = \frac{g(d)}{n(d) \cdot \rho(d)} \quad \text{with} \quad \alpha \leqslant 1$$

$$f(d) = \beta \cdot \pi \cdot d^2 = \frac{6}{\gamma \cdot d} \cdot \frac{g(d)}{n(d) \cdot \rho(d)} \quad \text{with } \beta \geqslant 1 \tag{12}$$

$$\gamma = \frac{\alpha}{\beta}$$

$$n(d) = \frac{6}{\alpha \cdot \pi} \cdot \frac{g(d)}{d^3}$$

the factors λ and ν are generally given by

$$\lambda = \frac{\alpha}{6 \cdot \beta} = \frac{\gamma}{6} \quad \text{and} \quad \nu = \frac{\alpha \cdot \pi}{6}. \tag{13}$$

Table 1 presents the degree of fineness $F_S = 1/\overline{d}$ and $F_G = 1/d^{-3}$ and of uniformity $G_S = 1/\sigma_F$ and $G_G = 1/\sigma_N$ for the distributions of granulation in Figure 1.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Hansel, N. 1968a. Nichtbindige künstliche Korngemische mit vorgegebenen Parametern und ihre Anwendung bei systematischen Versuchen Wiss. *Zeitschrift der TU Dresden,* vol. 17., no. 2, p. 449-52.

——· 1968b. Uber die Geschiebeaufnahme unterhalb von Flachlandspeichern Symposium "Aktuelle Fragen der Flussregulierung und der Feststoffführung".

Kezdi, A. 1964. *Bodenmechanik,* vol. 1.

Kneschke, A.; Schoch, M. 1959. Über die Auswertung allgemeiner Siebrückstandskurven, Freiberger Forsch, Reihe A, vol. 130, p. 11-25.

Bedload measurement by means of bottom plates and bedload samplers with hydrophone attachments

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Abstract. In view of the great importance attached to a thorough knowledge of bedload transport and solid discharge, efforts are made in many countries to improve bedload samplers and bedload measuring methods.

At the Federal Institute of Hydrology, a device has been developed recently and tested which makes it possible simultaneously to measure the quantity of bedload transport and to observe it acoustically.

For this purpose, a bedload sampler has been fitted with an underwater microphone, the sound transmitter of which is connected, via an amplifier, to a headset and a loudspeaker. The latter is designed for three sound frequencies which can be turned on either singly or together, as required.

In addition to a description of the device, the first measurements are presented which also show the irregularities in bedload transport caused during the passing of a ship, in particular during the so-called critical runoff phase.

MESURE DU CHARRIAGE AU MOYEN DE PLAQUES DE FOND ET D'UN DISPOSITIF COLLECTEUR UTILISANT UN MICROPHONE

Résumé. L'importance qu'on attache à une connaissance précise des transports et des charges des galets pour en déduire des considérations hydrologiques et faire des prévisions (par exemple érosion du fond ou élévation du fond) conduit à se pencher attentivement sur les mesures hydrotechniques (par exemple dans la construction de barrages, dans le choix des emplacements d'accès aux ports ou aux canaux) ; c'est pourquoi, dans beaucoup de pays on cherche à obtenir une amélioration des dispositifs collecteurs et des techniques de mesure de l'entraînement des galets.

Il y a peu de temps, l'Institut fédéral d'hydrologie de Coblence a mis au point et testé un appareil susceptible d'évaluer la quantité de galets entraînés, et d'enregistrer *en même temps* la perception acoustique de l'entraînement de ces galets, à savoir : le roulement, le glissement et le saut des galets charriés.

A cet effet, le dispositif collecteur a été pourvu d'un microphone placé sous l'eau et dont l'émetteur était relié par l'intermédiaire d'un amplificateur à un écouteur et à un haut-parleur. Ce dernier est doté de trois fréquences de son qui peuvent être choisies soit séparément, soit ensemble.

Outre la description de l'appareil avec figures à l'appui, l'article donne les premiers résultats des mesures ; ils font apparaître par exemple des irrégularités dans l'entraînement des galets au passage des bateaux, notamment pour le débit dit critique.

MEDIDA DE ARRASTRE DE FONDO POR PLACAS Y MUESTREADORES CON HIDRÓFONO

Resumen. A la vista de la gran importancia que presenta un completo conocimiento del arrastre de fondo y caudal salido en relación con consideraciones hidrológicas y previsiones (p. ej. erosión de fondo y elevación) y de su significación con respecto a las medidas de ingeniería hidráuluca (p. ej.: la construcción de presas, selección de emplazamientos de entrada de bahía o puntos de derivación de canales), en muchos países se realizan esfuerzos por mejorar los dispositivos de toma de muestras de arrastre de fondo y los métodos de medición de ésta última.

En el Instituto Federal de Hidrología, se ha puesto a punto recientemente y se ha ensayado un dispositivo por medio del cual es ahora posible medir simultáneamente el volumen del arrastre de fondo, y observarlo acústicamente (la rodarura, deslizamiento y salto de los granos).

Con estos fines, se ha equipado un tomador de muestras de arrastre de fondo con un micrófono sumergido cuyo transmisor de sonido se conecta por medio de un amplificador al equipo de cabeza y a un altavoz. Este último está proyectado para tres frecuencias de sonido que se pueden aplicar una a una o juntas, según se desee.

Además de la descripción del dispositivo, se presentanlos primeros resultados de medición que también muestran, entre otras, las irregularidades del transporte del arrastre de fondo durante el paso de un barco, en particular en la llamada fase crítica de escorrentía.

ИЗМЕРЕНИЕ ДОННЫХ НАНОСОВ ПОСРЕДСТВОМ ДОННЫХ ПЛИТ (ПЛАСТИН) И ДОННЫХ БАТОМЕТРОВ С ГИДРОФОННЫМ ПРИСПОСОБЛЕНИЕМ

Резюме. В связи с болшой важностью, которая придается полным знаниям о транспорте донных наносов и расходу твердого стока в связи с гидрологическими расчетами (напр, донной и боковой эрозии) и его значением для гидротехнических мероприятий, которые должны быть проведены (например, сооружение плотин, выбор местоположений для входа в гавань или точек разветвления каналов), — во многих странах деляются попытки усовершенствования батометров и методов измерения донных наносов.

В Федеральном институте гидрологии недавно было разработано и испытано устройство, посредством которого можно **одновременно** измерить транспорт донных наносов объемным способом и наблюдать его акустически (перекатывание, скольжение и подпригивание частиц). Для этой цели батометр был оборудован подводным микрофоном, передатчик звука которого связан через усилитель с громкоговорителем. Последний рассчитан на три звуковых частоты, которые могут по желанию воспроизводиться раздельно или вместе. Первые результаты испитаний устройства показывают беспорядочность в движении донных наносов во время прохождения судна, особенно во премя так называемой критической фазы стока.

INTRODUCTION

Knowledge of bedload travel characteristics plays an important part in hydrology for calculations and forecasts (e.g. bottom erosion or bottom rise). It is also important in hydraulic engineering (e.g., the construction of dams, the selection of the proper location for harbour entrances and canal functions and maintenance dredgings).

This knowledge should comprise :

1. The start of bedload movement as a function of: stage, water depth, discharge, flow velocity, river cross-section, length and radius of river bends, location of hydraulic structures, number and draught of ships travelling up and down the river:
2. The quantity of bedload material moved per unit time, including the maximum, minimum and mean values for specific conditions and for various widths of the river;
3. The same as 2, but for several longitudinal sections;
4. The fluctuations for short and long durations;
5. The size and shape of grains and the distribution of grain sizes.

In calculating erosion by means of empirical formulae, certain measured values are required such as the magnitude of the critical discharge at which bedload transport

begins and the amount of bedload transport that prevails at the cross-section concerned under heavily fluctuating stage conditions.

Efforts are made, therefore, in a number of countries to improve both bedload samplers and bedload measuring methods.

The most important measuring instrument is, without doubt, the bedload sampler. It can be complemented by a number of other measuring procedures, among them the acoustic method which the Federal Institute of Hydrology has recently developed and tested. The instruments are described below and illustrated in Figure 1.

BOTTOM PLATE WITH HYDROPHONE ATTACHMENT

The device shown in Figure 2 is used to detect movement of any coarse-grained bedload material. It consists of a brass plate 150 mm wide, 550 mm long and 5 mm thick, fitted to a frame weighing about 100 kg and is lowered to the bottom of the river from one of the booms of a ship. The plate rests flat on the river bottom, transverse to the direction of flow. Attached to the brass plate is a hydrophone which picks up the sound caused by the bedload grains rolling, sliding, tumbling or springing across the plate, transmitting it to earphones or loudspeakers via an insulated two-wire cable and an amplifier (220 V a.c.) arranged aboard the ship. In addition, a graphic recorder registers the acoustic information. This recorder is not an absolute necessity, but it is extremely useful as it allows detection and recording of the fluctuations in bedload transport, especially in the case of long-term measurements. This is of particular importance where the investigation covers a large area.

With the bottom plate, the following measurements concerning bedload transport have been carried out so far:

1. Determination of the so-called critical discharge at several stations in the Upper Rhine and in the Moselle river;
2. Determination of the influence of shipping activities on the Upper Rhine.

In the Upper Rhine, for example, it was found that at certain low water levels, the bedload material moved only when a ship passed by, and even this only when the bow and the stern were passing.

In another example, it was noted that, on 2 March 1970, near Lehmen, when the Moselle carried flood water and all shipping had ceased, only a few bedload stones tumbled across the plate every 20 to 30 sec. for a duration of about 3-5 sec. This means that there was no continuous bedload travel. The bottom flow velocity measured simultaneously, but separately, with the aid of a flow meter was 80-100 cm/sec. All of the moving bedload material was flat in shape, typical dimensions being 10 mm × 50 mm × 120 mm (bedload was measured separately by means of a bedload sampler).

A BEDLOAD SAMPLER WITH HYDROPHONE ATTACHMENT

Figure 3 shows the "Arnhem"-type of bed-load sampler with hydrophone attachments.

In rivers with a heavy bed-load transport, the movement of the bedload (as far as it is of coarse grains) into the sampler, and thus all fluctuations in bedload travel, can be easily heard.

Another advantage of the hydrophone attachment consists of the possibility of checking on whether or not the sampler changed its position during the time (5 min.)

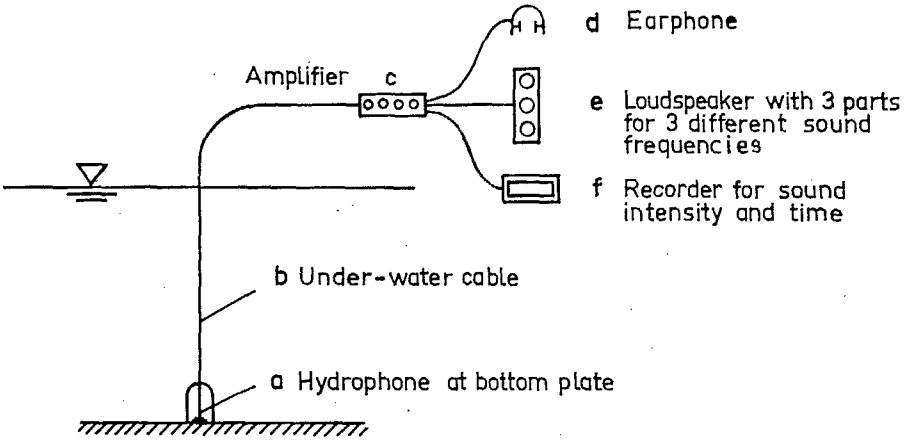


Figure 1 – *Pattern of basic instruments arrangement*

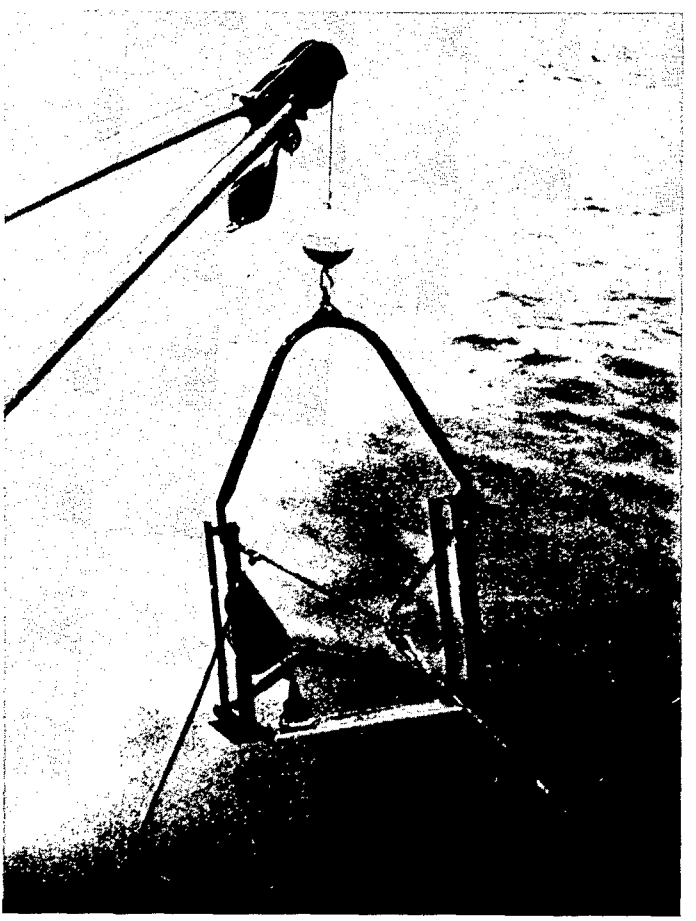


Figure 2 – *Bottom plate with hydrophone attachment*

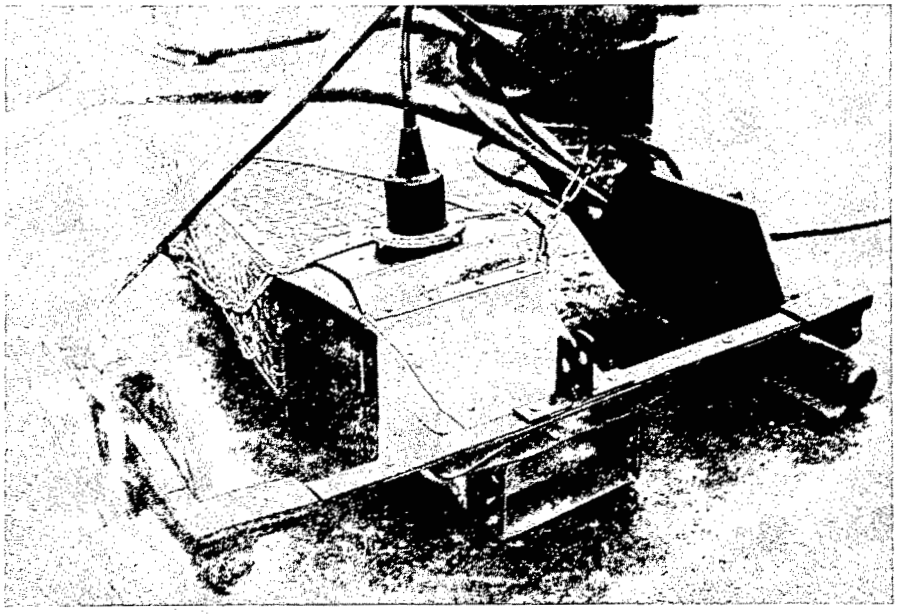


Figure 3 – *"Arnhem-type" bedload sampler with hydrophone attachment*

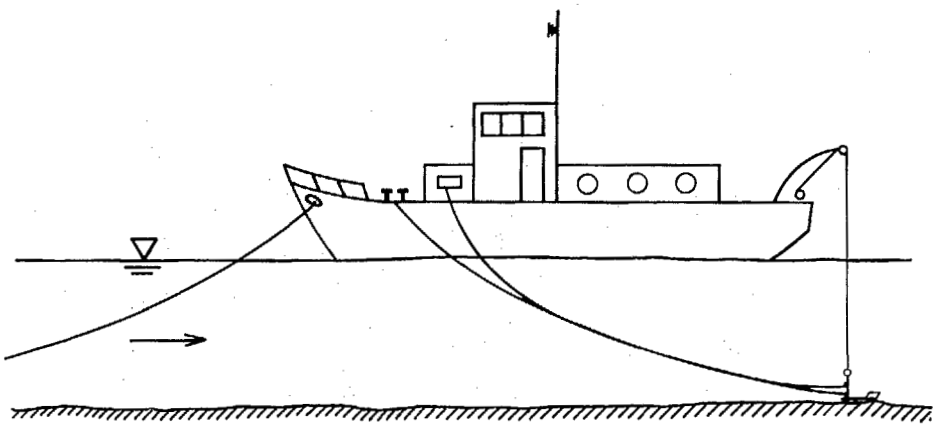


Figure 4 – *Arrangement for holding ropes on board the measuring ship for acoustic devices*

it rested on the river bottom. Such a change in position may occur when the anchors of the measuring ship drag under flood conditions, or when the measuring ship is displaced laterally by passing traffic.

If the bedload sampler placed on the river bottom is displaced, the shifting motion can be clearly detected acoustically and will, in some cases, confuse the measurement. As soon as the displacement has been recognized, a complementary measurement can be carried out. In this connexion it may be pointed out that, in any case, the Federal Institute of Hydrology performs at least three measurements for every bedload measuring point of the 5 to 7 verticals in the cross-section.

With the aid of the hydrophone, it is also easy to ascertain the moment at which the bedload sampler comes to rest on the river bottom when lowered for sampling. As a result, it is then possible to prevent the two sampler holding ropes (Figure 4) from being excessively slackened. This is important because if the ropes are slackened too much, the sampler may drift too far and may then "scrape" bottom material into itself when being retrieved, thus invalidating the measurement.

SUMMARY

The acoustic measuring instruments, their methods of use and the results from them have been described very briefly.

The tests carried out so far have shown that it may be appropriate either to actually measure bedload transport acoustically (by means of a hydrophone attached to a bottom plate), or to check the measurement acoustically (by means of a hydrophone attached to the bedload sampler).

Each of the methods described involves the use of a hydrophone and can be applied independently of the other.

The instrumentation costs are comparatively low and amount to a total of only about DM.1,200 – for the hydrophone, 20 m of cable, the amplifier, the earphones and the loudspeaker. To this must be added the cost of the frame with the bottom plate, or the cost of the bedload sampler.

It is intended to continue these tests and to use these instruments not only from on board ship, but also –perhaps as modified versions– from bridges, using the measuring van of the German Water and River Navigation Authorities. This van makes it possible to carry out, with the same crew and with the same instrument, on one and the same day, a large number of measurements at widely scattered points.

DISCUSSION

Struijk: Did you try to control the distribution of bedload above sand ripples?

With what kind of instruments did you measure the velocity at the bottom?

Hinrich: We have made measurements in the Rhine and Moselle where we did not find any ripples. I assume you are thinking of tidal areas. When measuring the bedload we are measuring at the same time the discharge and the distribution of velocity in several verticals. Additionally, the velocity is being measured at a distance of 15 cm from the bottom with a current meter.

Kinori: 1. What could be done for calibration of the sounds?

2. Could the sound be recorded on a tape recorder and the reading evaluated later?

3. The acoustic method has the advantage of being a continuous measuring device.

Hinrich: The method of acoustic measuring is not new. Türk used microphones more than two decades ago to detect the bedload movement. Further attempts have been made with devices developed by Farenholtz.

Your main question concerns the calibration. This question needs attention, and I should like to refer to the paper of Dr. Burz. There is perhaps a certain parallel between the measurement of turbidity for the suspended load and the determining of the weight with the help of pictures. We hope, in the course of time, to obtain results by interpreting the volume and frequency of the tune. The Federal Institute of Hydraulic Engineering at Karlsruhe has used the hydrophone for continuous measurements.

De Vries: A hydrophone can be used without a plate but with a sounding weight (streamlined), and can be located near the bottom, but not necessarily resting on the bottom. By having the hydrophone 15 or 20 cm above the bottom of the river-bed, the noise of the bed movement could be detected. This would have the advantage that it would not slide along the bottom at high flows. Have you tried this method and if so, would you please comment?

You mentioned that one hydrophone has been designed for three sound frequencies. What were these frequencies? And would you please comment on the advantages or disadvantages of using one frequency instead of another. For example, have you found, perhaps, that the noise created by water turbulence around the sampler was of a certain frequency and could be eliminated by setting the hydrophone for a different frequency?

Hinrich: When Dr. Türk installed a hydrophone in the water independent of the bottom, he found that the sound interference of the water turbulence and of the screws of ships was so great that no useful measurements could be taken. The best results are obtained when the bedload rolls directly over the plate.

As to the frequencies, we have tried out three different ones. In the middle area you can hear nearly all essential sounds, but this method is still in a rather early phase of development.

De Vries: Which modifications were applied to the original Arnhem sampler and how did you calibrate the modified instrument?

Hinrich: The improvements of the Arnhem sampler partly concern the texture. With a width of mesh of 1.4 mm you can even measure sand smaller than 1.4 mm. In order to protect the texture from detritus, we have added a further mesh of 30 mm width. This new arrangement has been in use for the past three years without interruption.

Another improvement concerns the underside of the sampler, which now contains an aluminium flap (patent Tippner) allowing more rapid unloading of the bedload. This reduces the time of the whole procedure. There is no change in the general shape.

De Vries : But did you change the dimensions?

Hinrich: No, the opening is 8 by 16 cm. We did not change the dimensions since other institutes are using the same device.

Burz: Did you find differences between the distribution of grain in a bank to the bedload moving nearby?

Hinrich: No, there was complete conformity.

Oebius: How do you define the beginning of bedload movement? Do you have a definition of your own, do you follow the usual definitions?

Hinrich: The movement begins at the spot with the maximum velocity and depth of water. The beginning of movement takes place when the grains start rolling. However, it is only effective if it occurs on a broadstrip.

Oebius: No, this definition is not usual. You mentioned that the Moselle in a flood moves big grains which you can detect with your hydrophone. In the event of these grains only occasionally passing, this would not be a movement according to the definition of Schiels.

Hinrich: When we observe that single grains start rolling, we are certainly in the area of the critical discharge, and I have to begin measuring, integrating over a time period of not 1 min or 3 min as in a real discharge, but 5 or even 15 or 30! But I think that, in fact, the movement of the first grains is the beginning of bedload transport.

Oebius: Related as usual to 1 minute, you would get a transport of 0.1 grans per sec.?

Hinrich: Yes. Even low figures of transport amount to considerable sums if multiplied by the 86,400 sec. of a day and the width of the bottom.

Oebius: Could you distinguish clearly the sound of a grain from that of a screw of a ship?

Hinrich: Yes, we are fully convinced of it, and it has been proved repeatedly.

I should like to make an additional statement. Last week we observed the movement of bedload by submarine television in the Rhine, and there was no influence of the turbidity on the quality of the picture. Up until now the movement was always a secret, particularly the way in which the grains hopped into the sampler. We can now compare reality with the models. We could see precisely that in the zone of critical discharge the large grains remained in their position, but the smaller ones started moving in the gaps. The company which installed the television has assured me that the results can also be put on film for use later.

Field studies of sediment movement using fluorescent tracers

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Abstract. Particles of various diameters and specific gravities, coated with fluorescent dyes, were used to study sediment movement in sand-channel streams of the south-western United States. Simple and inexpensive techniques were developed for coating and injecting particles and for obtaining and analysing samples. The steady-dilution or time-integration methods are recommended for high velocity, flat bed conditions, while the spatial-integration method is more suitable for dune bed conditions. The transport rate, calculated by the time-integration method for a flat bed, was about 14 per cent greater than the measured transport rate. Particle velocities for the flat bed depended on size and density, whereas for the dune bed, particle velocities approximated to the dune migration velocity and appeared to be only weakly dependent on density.

ETUDES SUR LE TERRAIN DE MOUVEMENTS DE SEDIMENTS UTILISANT DES TRACTEURS FLUORESCENTS

Résumé. Des particules qui diffèrent par leur diamètre et leur poids spécifique sont enduites de teintures fluorescentes et utilisées pour étudier le mouvement des sédiments dans les courants à fonds sableux, dans le sud-ouest des Etats-Unis. Des techniques simples et peu coûteuses ont été développées pour enduire et injecter les particules ainsi que pour obtenir et analyser des prélèvements. Les méthodes dites "à dilution en régime permanent" et "à intégration dans le temps" sont recommandées pour les grandes vitesses sur un fond plat, tandis que la méthode "d'intégration dans l'espace" est plus appropriée aux lits en forme de dune. Le taux de charriage, calculé par la méthode d'intégration dans le temps pour un fond plat, était d'environ 14 % supérieur au taux de charriage mesuré. Les vitesses des particules, pour un fond plat, dépendaient de leur taille et de leur densité tandis que pour un lit en forme de dune, ces vitesses étaient approximativement celles du déplacement de la dune et elles dépendaient apparemment très peu des densités.

ESTUDIOS DE CAMPO DE MOVIMIENTO DE SEDIMENTOS, USANDO TRAZADORES FLUORESCENTES

Resumen. Se utilizaron partículas de diversos diámetros y pesos específicos, revestidas con tinturas fluorescentes, a fin de estudiar el movimiento de sedimentos en cursos de agua arenosos de la zona del sudoeste de los Estados Unidos de América. Se pusieron a punto técnicas simples y baratas para revestir e inyectar partículas y para obtener y analizar muestras. Los métodos de dilución continua o de integración se recomiendan para velocidad elevada y cauces planos, mientras que el método de la integración espacial es más adecuado para condiciones de cauce rugoso-arenoso. El valor de transporte, calculado por el método de integración para un fondo plano, era aproximadamente un 14 por ciento mayor que el valor de transporte medido. Las velocidades de partícula para los cauces planos dependían del tamaño y densidad, mientras que para los cauces rugoso-arenoso, las velocidades de partículas se aproximaron a la velocidad de migración de la arena y parecieron depender únicamente, en débil medida, de la densidad.

ПОЛЕВЫЕ ИССЛЕДОВАНИЯ ДВИЖЕНИЯ НАНОСОВ ПУТЕМ ИСПОЛЬЗОВАНИЯ ФЛЮОРОСЦЕНТНЫХ КРАСИТЕЛЕЙ

Резюме. Частицы различных диаметров и удельных весов, покрытые флюоросцентными красителями, использовались для изучения движения наносов в потоках с песчаными ложами на юго-западе США. Простая и недорогая методика была предложена для покрытия и ввода частиц и для получения и анализов проб. При высоких скоростях и плоском дне рекомендуются методы постоянного раствора или интеграции во времени, тогда как для условий грядового дна более пригодным является метод интеграции в пространстве. Скорость движения наносов, вычисленная с помощью метода интегрирования во времени для плоского русла, была на 14 % больше измеренной. Скорости движения частиц на плоском дне зависели от их размера и плотности, тогда как для грядового дна скорости частиц приближались к скоростям миграции гряд и лишь незначительно зависели от плотности.

INTRODUCTION

The applications of fluorescent tracers in routine sediment-discharge measurements have been investigated by the United States Geological Survey for several years. In addition, these techniques have been applied in special field research on the effects of particle density and size on sediment movement. This paper summarizes some of the highlights of these continuing investigations, with particular emphasis on applications to routine determinations of sediment discharge in streams with beds composed of fine sand.

THEORY

Three methods for determining sediment discharge from tracer techniques are the (1) steady-dilution method, (2) time-integration method, and (3) spatial-integration method.

In the *steady-dilution method,* a mixture with tracer concentration C_1 is injected continuously at position x_1 at the rate Q_T. By the conservation of mass,

$$Q_T C_1 = (Q_T + Q_S)\, C_2 \tag{1}$$

where C_2 is the equilibrium concentration at the downstream measuring section, x_2, and Q_S is the sediment discharge of the stream shown in Figure 1. It is assumed that

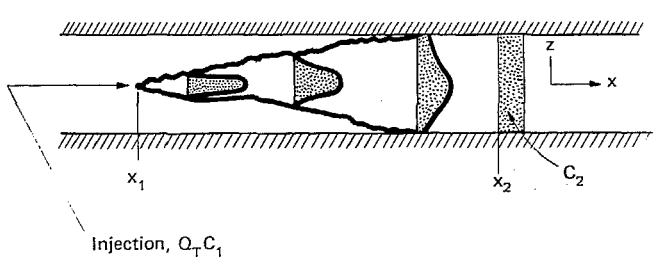


Figure 1 – *Definition sketch for steady-dilution method. Shade-area represents concentration distribution*

the background concentration of tracer in the stream sediment is negligible and that complete lateral mixing has occurred. Concentration is expressed as weight of tracer per weight of injected or sampled mixture. If $C_1 = 1$ and Q_T is much smaller than Q_S, the sediment discharge is given by

$$Q_S \approx Q_T/C_2 \tag{2}$$

where C_2 is independent of time, t, and lateral position, z. The concentration C_2 is determined by sampling at frequent time intervals and plotting C versus t (Figure 2). If C_2 is not independent of z, samples are collected continuously at several points in the cross-section to determine an average C_2.

Equation 2 is applied separately to each size class of interest and the results are summed to get total sediment discharge.

In the *time-integration method*, a weight, W, of tracer material is introduced as an instantaneous point or line source and samples are collected continuously at some cross-section downstream to yield the concentration-time curve of Figure 3.

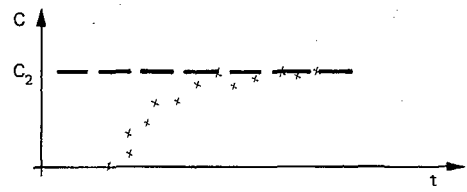


Figure 2 – *Determining* C_2

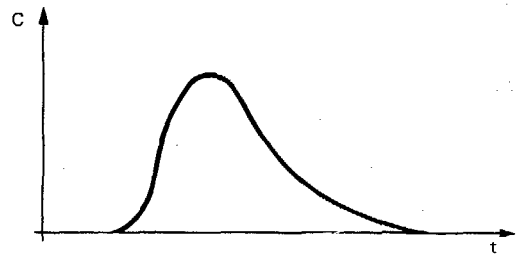


Figure 3 – *Concentration time curve for time-integration method*

By mass continuity,

$$W = Q_S \int_0^\infty C \, dt. \tag{3}$$

The quantities Q_S and $\int C \, dt$ are assumed to apply across the sampling section and Q_S should be reasonably steady during the sampling period. In practice, it may be necessary to integrate in the y and z direction so that C is the average concentration in the cross-section.

For the *spatial-integration method*, a quantity of tracer material is introduced as an instantaneous point or line source at position x_0 and time t_0 in Figure 4. The tracer cloud disperses downstream as shown, so that its centroid is at position $\bar{x}_1$ at time t_1 and $\bar{x}_2$ at time t_2. The velocity V of the tracer cloud is

$$V = \frac{\bar{x}_2 - \bar{x}_1}{t_2 - t_1} \tag{4}$$

and the sediment transport rate is

$$Q_S = V\, d\, \gamma_S (1 - \lambda) B \tag{5}$$

where d is the mean depth of movement of the tracer material in the bed, determined from core samples, B is the channel width, γ_S is the specific weight of the bed material, and λ is porosity of the bed. The centroid position, $\bar{x}$, is given by

$$\bar{x} = \frac{\int_0^\infty C x \, dx}{\int_0^\infty C \, dx} \tag{6}$$

The concentration C is assumed to apply uniformly in the cross-sectional area Bd. Usually, complete vertical and lateral mixing is very difficult to obtain, and the position of the tracer cloud must be defined by a grid or network of core samples. An average concentration C can then be determined and plotted against position x (Fig. 5) to approximate by summations the integrals in equation 6. Note that both the time-integration and spatial-integration methods are total recovery methods ; that is, if no tracer material is lost, the area under the C versus x curve is constant for different t values, and the area under the concentration-time curves is a constant for various positions x along the channel.

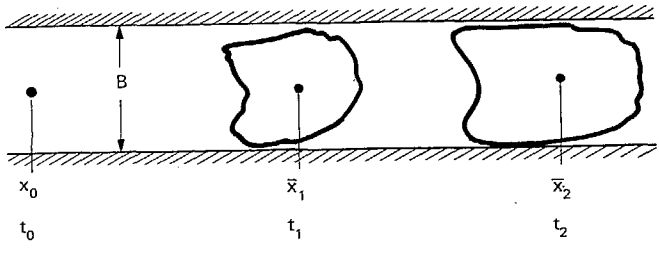


Figure 4 – *Definition sketch for spatial-integration method*

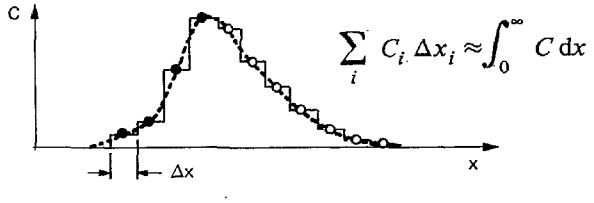


Figure 5 – *Approximation by summation of the area under the concentration-distance curve*

In all three methods, it is assumed that the properties of the tracer material are identical to the properties of the natural sediment, and that the amount of tracer material injected is small in comparison with the natural sediment transport rate.

TECHNIQUES AND EQUIPMENT

Application of tracer techniques requires four major steps: (1) preparation of tracer material, (2) injection, (3) sampling, and (4) analysing the samples.

1. Preparation of tracer material. Tracer material may be natural bed material from the stream of interest or commercially available graded sand and gravel. For quartz material, we have used both natural sand and commercial Ottawa sand. The commercial sand is pure and clean and coats readily with the fluorescent dye, but may be too well-rounded to match the hydraulic properties of stream sediments. The stream sediments were mostly quartz and feldspars, but were clean and reasonably free of heavy minerals. These materials also coated readily. For our experiments on density effects, we coated garnet (density 4.1) and monazite (density 4.8) from natural placer deposits, and lead shot and lead flakes (density 11).

To coat the material, about 500 cc of vinyl plastic powder was dissolved in 4 *l* of commercial grade acetone, and the dye powder was then added slowly with constant stirring until the solution was saturated. About 500 cc of the dye powder was required. The vinyl makes the dye coating less brittle and more resistant to abrasion, so that it does not flake off. The sediment to be coated was placed in a cement mixer and the dye solution was added slowly in increments by means of a plastic spray bottle, allowing time for the sand to dry completely between additions of dye. About 50 kg of quartz, requiring about 1 *l* of the dye solution, can be coated in about 15 min. with this technique. For small quantities of tracers, the dye can be applied by hand mixing in a shallow container.

Under UV light, red, yellow, blue, and green dyed particles can be distinguished easily by eye. For a single colour, yellow is recommended. If counting is to be done by an automatic counter, it is necessary to select dyes whose colour spectra do not overlap.

2. Injection. Coarse sediments can be injected dry at the water surface. Fine sediments injected dry will attach air bubbles and float, so sand-size material should be injected by mixing into a slurry with water and a small amount of detergent or wetting agent, and introducing the slurry near the stream-bed by pumping it or simply by pouring it into a large diameter tube and washing it down with stream water. The latter method is simple and inexpensive, requiring no special equipment, and is satisfactory for most practical applications in shallow streams.

3. Sampling. If the stream-bed configuration is dunes, the spatial-integration method is recommended, and the centroid of the cloud of tracer particles and mean depth of movement in the bed can be determined from a grid of core samples. Our core sampler, Figure 6a, was 7 cm in diameter and could sample through a depth of 60 cm. Each core was split into 5 equal increments to determine the depth of movement of sediment.

When the stream velocity is high and the bed configuration is flat, the sediment moves in a thin layer along the bed at relatively high velocities, and the continuous-feed or time-integration method should be used. Samples can be obtained by

bed-load samplers, or by a simple "dustpan" sampler shown in Figure 6b. This is a simple expansion chamber about 40 cm long with an entrance 2.5 cm high and 8 cm wide and with a small baffle 15 cm downstream of the entrace behind which the sediment settles. The capacity of the sampler is about 1 kg. Sampler efficiency is of no concern, as we are interested only in the ratio of marked to unmarked particles. Samples are washed into sample bags using stream water and a pressure spray can. Necessary equipment for dustpan sampling is shown in Figure 6c.

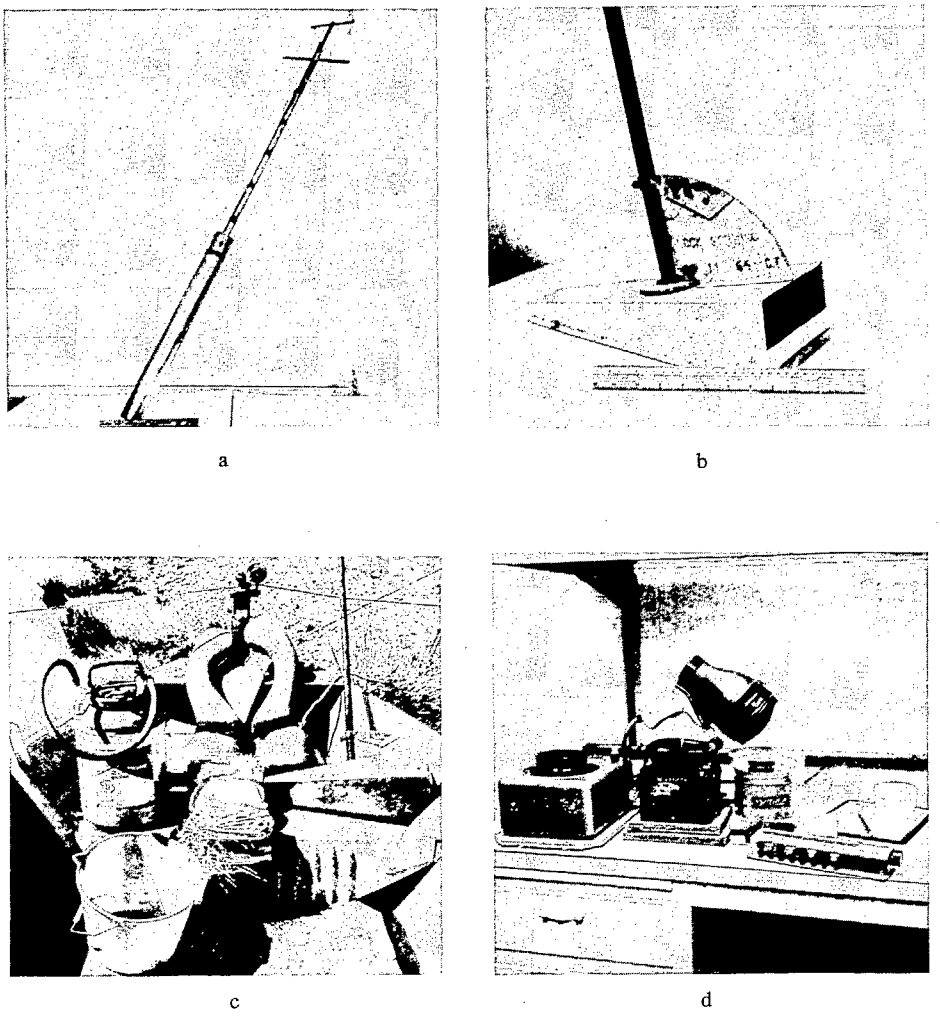


Figure 6 – *Equipment for sampling and analysing samples:*
Top left: (a) Core sampler
Top right: (b) "Dustpan" sampler
Bottom left: (c) Equipment for sampling
Bottom right: (d) Counting equipment
(The scale in a and b is 30 cm long.)

4. Sample analysis. Analysis of samples consists of sieving them into suitable-size fractions and determining the concentration of tracer particles in each size class. For dilute concentrations, the sample can be spread under a UV light and all particles counted. For more concentrated samples, sufficiently accurate determinations of the concentrations are obtained by counting approximately 400 tracer particles and weighing the amount of sample in which they are contained. The number of tracer particles are converted to weight by a predetermined calibration of number of particles per gram for each size class of interest. In some of our investigations, we counted 4 colours simultaneously, using a variable-speed Syntron parts feeder that could feed a continuous stream of sample particles through the UV light at any desired rate and a hand tabulator with 4 registers for recording counts of the 4 colours. The equipment used for counting is shown in Figure 6d. Of course, elaborate automated counting devices are available (Teleki, 1967 ; de Vries, 1968), but for investigations of limited scope or budget, the only necessary equipment is an UV light and a balance.

SOME APPLICATIONS AND RESULTS

Fluorescent tracer techniques were applied to studies of lateral, longitudinal, and vertical dispersion of particles in the bed, to studies of effects of size and density on rates of particle movement, and to determinations of sediment discharge for both the flat bed and dune bed configurations. Detailed results of the flat bed study will be published elsewhere (Rathbun, *et al.,* in press), and results of the dune bed study are currently being analysed. Here we will summarize only a few salient points of the studies.

Table 1 summarizes results of one sediment discharge determination for the flat bed configuration, where the total sediment load was computed using the time-integration method. The measured load shown in the table was determined from width-depth integrated samples at an artificial control structure constructed so that the entire sediment load passed in suspension over a sampling lip and was sampled with conventional suspended-sediment sampling equipment. Within individual size classes there are discrepancies of almost a factor of 2, but over-all agreement is considered quite satisfactory. This study was conducted in a straight uniform channel 22 m wide, with flow velocity of 1.24 m/sec and depth 0.76 m. The median diameter of the channel bed material was 0.25 mm, the bed was flat, the water discharge was about 21 m^3/sec, the water-surface slope was 7.0×10^{-4}, and the water temperature was 2° C.

Table 1. *Summary of calculations of the sediment-transport rate*

Sieve class (mm)	Q_S, calculated (metric tons per day)	Q_S, measured (metric tons per day)	$\frac{Q_S \text{ (calculated)}}{Q_S \text{ (measured)}}$
0.125-0.177	779	1 080	0.72
0.177-0.250	1560	850	1.84
0.250-0.350	678	647	1.05
0.350-0.500	107	184	0.58
0.500-0.707	33	23	1.44
0.707-1.00	6.3	0	—
Total	3 160	2 780	1.44

The flat bed study provided interesting information on mean particle velocity as a function of size and density (Fig. 7). The mean particle velocity is the velocity of the centroid of the cloud of tracer particles as the cloud migrated downstream (see Fig. 4). For the quartz, the minimum particle velocity occurred in the size class containing the median diameter of bed material, and the particle velocities varied from about 2 to 15 per cent of the mean water velocity. Particle velocities of garnet and monazite were much smaller and did not exhibit the pronounced minimum of the quartz. The variation of particle velocity with fall diameter (Fig. 7) is in accord with the findings of Sundborg (1956) and Meland and Norrman (1966), who discuss the tendency for large particles to move with large velocities.

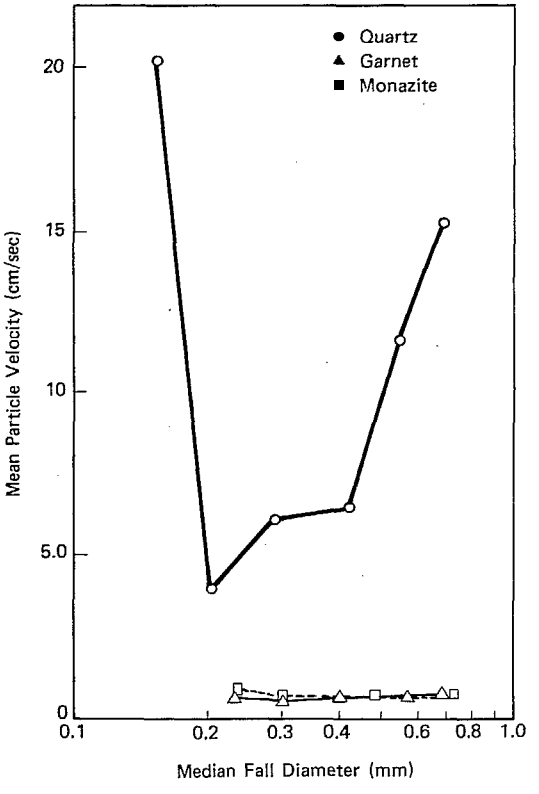


Figure 7 – *Variation of particle velocity with fall diameter*

Finally, a comparison of mean particle velocity for the flat bed and dune bed configurations provides an interesting insight into the different mechanisms of transport for the two bed forms. The spatial-integration method was used in the dune bed study and, although analysis of the data has not been completed, some preliminary centroid velocity values have been calculated. Table 2 shows the centroid velocities calculated from equation 4 for groups of injected particles for flow over a dune bed, with mean water velocity of 0.51 m/sec and mean depth of 0.76 m. The water temperature was about 15°C and water-surface slope and particle size were similar to those given above for the flat bed.

Table 2. *Summary of mean particle velocity, dune bed*

Mineral	Fall diameter (mm)	Centroid velocity (m/day)			Weighted mean velocity (m/day)
		After 7 days	After 36 days	After 74 days	
Quartz	0.323	11	9.9	4.5	7.3
Garnet	295	9.5	5.3	7.0	6.6
Monazite	303	9.4	6.9	4.2	5.6

Two important facts emerge from consideration of the data in Table 2 and Figure 7. First, the mean particle velocity is several orders of magnitude larger for the flat bed condition than for the dune bed, and second, mean particle velocity is only very slightly dependent on particle density for the dune bed condition. These observations indicate the basic difference in transport mechanisms for the two bed forms. For the flat bed, material is transported in a continuous thin sheet near the bed, with essentially no mixing of the moving particles and the bed. For the dune bed, particle mixing occurs to a depth equal to the elevation of the dune troughs, a great amount of material is in motion, but it moves at about the same velocity as the migration velocity of the dunes.

SUMMARY AND CONCLUSIONS

The fluorescent tracer technique is a versatile, powerful tool for studying the movement of sediments. The techniques for dyeing and injecting tracer particles and for sampling and analysing samples from shallow sand-bed streams is simple, inexpensive, and requires little specialized equipment. The steady-dilution method or time-integration method is recommended for high-velocity, flat-bed conditions, while the spatial-integration method is preferred for the dune bed case.

Results from two field studies demonstrate the applicability of these techniques to determining sediment discharge and to defining some effects of particle size and density on mean rates of particle movement.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Meland, N.; Norrman, J. O. 1966. Transport velocities of single particles in bed-load motion. *Geog. Annaler,* vol. 48, ser. A, p. 165-82.

Rathbun, R. E.; Kennedy, V. C.; Culbertson, J. K.; in press. Transport and dispersion of fluorescent-tracer particles for the flat-bed condition, Rio Grande conveyance channel, New Mexico. *U.S. Geol. Survey Prof. Paper.*

Sundborg, A. 1956. The river Klarälven–a study of fluvial processes. *Geol. Annaler,* vol. 38, p. 127-316.

Teleki, P. G. 1967. Automatic analysis of tracer sand. *J. Sed. Petrol.,* vol. 37, n° 3, p. 749-59.

Vries, M. de 1967. Photometric counter for fluorescent tracers. *La Houille Blanche,* vol. 22, no.7, p. 717-22.

DISCUSSION

De Vries: The models used by Dr. Nordin for the interpretation are relatively simple, but one must realize that these models can only be applied under very favourable conditions: (a) large sediment transports (example I); (b) nearly constant discharge over a long period (example II, for about 30 to 40 days).

In Nature, these conditions are usually not fulfilled and, therefore, these models do not apply. Thus more complicated models are necessary.

Nordin: I agree with you completely. In one of these channels the flow was very uniform. In the case of variable flow conditions in variable cross-sections, there is no easy way to determine sediment discharge.

Douillet: Comment avez-vous défini la distance nécessaire entre la section d'injection et la section de prélèvement ? Cette distance étant la distance nécessaire pour qu'il y ait bon mélange, quel était l'ordre de grandeur de cette distance pour les mesures que vous avez effectuées ?

Nordin: In the special inauguration method we take a few preliminary samples and analyse them with a total intensity typemeter or define the outline of our cloud. This is the basis for a more detailed sampling scheme. In the case of the time-integration and constant-feed methods, we did not develop a criterion for the mixing length. A mixing length of less than 20 channel widths was not satisfactory and it was not necessary to exceed 200 channel widths downstream, even under the most favourable conditions.

Struijk: In co-operation with the Delft Hydraulic Laboratory, we have sampled probes in the rivers of the Netherlands this year. We have obtained very good experience with frozen-in-tracers in a piece with a diameter of 30 cm and a height of 8 cm. In this case the dyes are frozen in and the grains are slightly dissolved and follow the water. The velocity of the water was about 1 m/sec.

Mesures de transports solides en suspension effectuées par l'ORSTOM en Afrique

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Résumé. Plusieurs hydrologues de l'Office de la recherche scientifique et technique outre-mer (ORSTOM) se sont livrés à des recherches méthodologiques sur les transports solides en suspension, au Togo, au Tchad et au Cameroun. Les principaux résultats auxquels ils ont abouti sont exposés dans la présente communication.

Des indications détaillées sont données sur le matériel et les techniques préconisées, en ce qui concerne les prélèvements d'échantillon en rivière, ainsi que les mesures de concentration et de granulométrie en laboratoire. Toutes les causes d'erreur sont passées en revue et l'on évalue leur importance relative.

Dans l'ensemble, les méthodes mises en œuvre par l'ORSTOM en Afrique conduisent à une précision satisfaisante (5 à 10 % en général), tout en étant peu onéreuses et ne nécessitant qu'un matériel assez léger.

MEASUREMENTS OF SUSPENDED SEDIMENT UNDERTAKEN BY ORSTOM IN AFRICA

Abstract. Several hydrologists of the "Office de la Recherche Scientifique et Technique Outre-Mer" (ORSTOM) have carried out methodological investigations on suspended sediment discharges in Togo, Chad and Cameroon. The main results achieved are given in this paper.

Precise details are given about the devices and processes recommended to sample water of rivers and to measure concentrations and grain-size distributions in the laboratory. Every cause of error is examined and its relative importance is evaluated.

Generally speaking, the methods used by ORSTOM in Africa give a reasonable accuracy (from 5 to 10 per cent), while being not very expensive and requiring no heavy material.

MEDIDAS DE SEDIMENTOS EN SUSPENSIÓN EFECTUADAS POR ORSTOM EN ÁFRICA

Resumen. Varios hidrólogos de la Oficina de Investigación Científica y Técnica en Ultramar (ORSTOM) han realizado investigaciones metodológicas de sedimentos en suspensión en Togo, Chad y Camerún. En la presente comunicación se exponen los principales resultados a los que llegaron. Se dan indicaciones detalladas sobre el material y técnicas utilizadas en lo que se refiere a la toma de muestras en orillas, así como a las medidas de concentración y de granulometría en laboratorio y se pasa revista a todas las causas de error, evaluando su importancia relativa.

En conjunto, los métodos aplicados por la ORSTOM conducen a una precisión satisfatoria (5 al 10 per ciento en general), y al mismo tiempo no son muy costosos y no precisan material pesado.

ИЗМЕРЕНИЕ РАСХОДОВ ВЗВЕШЕННЫХ НАНОСОВ

Резюме. Несколько гидрологов из Управления Научных и Технических Изысканий (ORSTOM) выполнили исследования взвешенных наносов в методических целях в республиках Того, Чад и Камерун. Основные результаты, полученные ими, излагаются в данном докладе.

Точные и детальные данные даны в отношении приборов и методики, рекомендуемых для отбора проб воды в реках и для измерения концентраций наносов и лабораторного определения распределения частиц. Исследо-

ваны возможные причины возникновения погрешностей и установлена их относительная величина.

Вообще говоря, методы, применяемые ORSTOM в Центральной Африке, дают хорошую точность (от 5 до 10 %), в то время как они сравнительно недороги и не требуют тяжелого оборудования.

Depuis 1965, le Service hydrologique de l'Office de la recherche scientifique et technique outre-mer (ORSTOM) a intensifié l'étude des transports solides, qui jusque-là n'avait pas encore été l'objet de recherches systématiques. Plusieurs hydrologues de l'ORSTOM, notamment J. Colombani au Togo et au Dahomey, B. Billon au Tchad et J. F. Nouvelot au Cameroun, se sont livrés à des recherches méthodologiques sur les transports solides en suspension. Les résultats auxquels ils sont parvenus ont déjà fait l'objet de publications partielles[1]. Notre propos ici est d'établir une première synthèse de ces recherches méthodologiques, qui ont été conduites avec le souci d'aboutir à des procédés pratiques peu onéreux et bien adaptés aux régions tropicales.

1. PRELEVEMENTS D'EAU EN RIVIERE

1.1. Méthode de prélèvement

Le matériel utilisé comprend, d'une part, le matériel classique utilisé pour les jaugeages de débit liquide :

Canot pneumatique (Zodiac Mark II ou Mark III) équipé d'un moteur hors-bord, ou passerelle pour les petits cours d'eau ;

Câble gradué pour repérer la position du canot dans la section de mesure, ou sur les cours d'eau très larges, cercle hydrographique et balises de repérage sur les berges ;

Treuil avec câble électro-porteur, moulinet et saumon (généralement saumon Ott 25 kg de type Sonas-Aras, éventuellement saumon de 50 ou même 100 kg), ou perche pour les petits cours d'eau ;

D'autre part, le matériel spécifique pour les prèlèvements d'eau :

Pompe à main de type Japy, reliée par un tuyau de caoutchouc à un tube de prélèvement fixé au saumon ;

Jeu de récipients de 10 litres, dans lesquels sont conservés les échantillons d'eau prélevés.

Nous ne nous étendrons pas sur le matériel classique de jaugeage, sauf pour préciser que le saumon doit, dans la mesure du possible, être d'un poids suffisant pour que l'angle de dérive du câble de suspension reste négligeable (inférieur à 10°), compte tenu du fait que l'adjonction du tube de prélèvement et surtout du tuyau de caoutchouc augmente sensiblement la dérive.

Le tube de prélèvement est fixé au saumon dans une position horizontale. Dans le cas du montage Sonas-Aras, il est soudé à une plaquette métallique, elle-même boulonnée sur la tige métallique de suspension du saumon. Un tuyau de caoutchouc, de 10 ou 20 mm de diamètre intérieur suivant les expérimentateurs, relie l'extrémité aval du tube de prélèvement à la pompe. Ce tuyau doit être assez épais et renforcé par une armature textile, pour qu'il n'ait pas tendance à s'aplatir pendant le

1. Voir bibliographie en fin d'article.

pompage. Pour éviter cet aplatissement, il faut limiter les pertes de charges et ne pas couder à angle droit le tube de prélèvement, surtout si l'on utilise une pompe Japy assez puissante (modèle n° 2). Un léger coude de 140 à 160° vers le haut est cependant admissible. La pompe Japy est fixée dans le canot à proximité du treuil (fixation sur le porte-à-faux métallique dans le cas d'un treuil Ott du type Newa).

Les récipients utilisés pour conserver les prélèvements d'eau sont constitués soit par des jerricans de 10 litres en plastique, soit par des dames-jeannes en verre de même contenance. Les jerricans nécessitent un transvasement à l'arrivée au laboratoire, mais ils sont beaucoup plus pratiques pour le transport que les dames-jeannes qui doivent être protégées contre les chocs (tressage d'osier ou panier). Les récipients sont repérés par un numéro marqué à la peinture.

Le matériel de prélèvement utilisé par les hydrologues de l'ORSTOM peut paraître assez rudimentaire par rapport aux échantillonneurs plus perfectionnés du genre de la "turbidisonde Neyrpic" ou des "sediment samplers". En fait, tout en étant d'un prix d'achat plus faible, il présente sur eux l'avantage de permettre des prélèvements plus importants, ce qui améliore la précision des résultats de mesure en réduisant très sensiblement l'importance relative des erreurs et pertes de matière solide au cours des manipulations de laboratoire. Cet avantage compense largement l'inconvénient d'un prélèvement moins correct du point de vue théorique.

L'appareillage de l'ORSTOM peut, en effet, à première vue, porter à critique du fait que l'eau pénètre dans le tube de prélèvement avec une vitesse accélérée ou ralentie par l'effet du pompage. Contrairement aux échantillonneurs classiques, notre appareillage ne comporte pas de dispositif spécial pour éviter de modifier la vitesse naturelle de l'écoulement au point de prélèvement.

Mais les échantillonneurs classiques visent à effectuer, en un temps donné, des prélèvements dont le poids de matière solide soit proportionnel au produit de la vitesse V du courant par la concentration C en matière solide.

Au lieu de mesurer directement ce produit $V.\ C$, que l'on peut appeler "densité de débit solide", notre appareillage vise seulement à mesurer la concentration C, la vitesse V étant mesurée indépendamment par la méthode classique du moulinet.

La question essentielle qui se pose est donc de savoir si la concentration C est sensiblement perturbée par la modification de la vitesse au point de prélèvement. Des essais systématiques à différentes cadences de pompage et des comparaisons minutieuses avec d'autres techniques de prélèvement devront permettre de répondre complètement à cette question.

Les premiers essais effectués tendent à indiquer que l'influence de la vitesse sur la concentration est faible et introduit des erreurs de mesure généralement très inférieures à 10 %. Il est d'ailleurs probable que cette influence est tout à fait négligeable pour les particules d'argile et de limon fin d'un diamètre inférieur à 0,02 mm. Elle ne commencerait à devenir appréciable que pour les limons grossiers et surtout pour les sables de diamètre supérieur à 0,1 mm.

Dans la majorité des cas, les éléments sableux ne constituent qu'une faible fraction des transports solides en suspension. Les échantillonneurs classiques prélèvent des volumes beaucoup trop réduits pour donner un échantillonnage valable de ces éléments. Si l'on cherche à les mesurer de façon précise, il est recommandé d'utiliser, en plus de notre appareillage, un échantillonneur à décantation du type "bouteille de Delft DF 12", mis au point par le Laboratoire d'hydraulique de Delft (Pays-Bas). Cet appareil présente l'avantage de permettre de prélever non pas quelques grains de sable isolés, mais plusieurs dizaines de grammes de sable en prolongeant la mesure aussi longtemps que nécessaire (10 ou 20 minutes par exemple).

Les essais systématiques relatifs à l'influence de la vitesse de pompage sur la concentration devront être conjugués avec un programme de mesures permettant de mettre en évidence les fluctuations de concentration liées à la turbulence de l'écoulement. Ces fluctuations encore mal connues sont sans doute susceptibles d'affecter la précision des mesures de façon non négligeable, si les échantillons sont prélevés trop rapidement. Pour cette raison la durée d'un prélèvement de 10 litres devrait être d'au moins une minute.

La méthode de prélèvement utilisée par les hydrologues de l'ORSTOM suppose, comme on l'a déjà vu, que les vitesses d'écoulement soient mesurées indépendamment au moulinet, ce qui est facile à réaliser en pratique puisque l'appareillage de prélèvement comporte tout le matériel nécessaire. Les mesures de vitesse sont faites en principe avec la même densité que pour les jaugeages classiques (en général 40 à 100 mesures réparties sur 10 à 15 verticales). Cette densité est supérieure à celle des prélèvements, comme on le verra en 1.2. Cependant, pour les petits cours d'eau de régime plus ou moins torrentiel, on peut être contraint de réduire la densité des mesures de vitesse, de façon que pendant la durée totale des opérations le niveau de l'eau ne varie pas de plus de quelques centimètres.

1.2. Densité des points de prélèvement

La densité des points de prélèvement dans la section de mesure est limitée par les possibilités pratiques de transport des échantillons. La durée des opérations de prélèvement et de dépouillement au laboratoire peuvent également imposer certaines limitations.

Dans beaucoup de cas, il apparaît que la concentration varie moins que la vitesse dans la section de mesure. Il n'y a donc pas d'inconvénient à admettre une densité de prélèvements sensiblement inférieure à la densité des mesures de vitesse.

Jusqu'à présent, les densités de prélèvements adoptées par les hydrologues de l'ORSTOM ont varié entre 12 et 32 points, répartis sur un nombre de verticales compris entre 5 et 8. Ces densités sont restées assez faibles pour des raisons essentiellement pratiques. De nombreux essais seront nécessaires pour évaluer le gain de précision procuré par une augmentation de la densité et pour déterminer la répartition optimale des verticales dans la section et des points de prélèvement sur chaque verticale.

Il sera sans doute difficile d'établir à ce sujet des règles générales suffisamment précises. Cependant, on peut déjà dire que le nombre de verticales doit être d'autant plus grand que le lit est large et irrégulier. A la suite des essais qu'il a effectués sur le Chari à Fort-Lamy (largeur de l'ordre de 400 m, profondeur maximale d'environ 10 m), B. Billon, par exemple, a estimé qu'un nombre de huit verticales était insuffisant, car il a observé des variations transversales de concentration assez rapides (30 à 50 % en quelques dizaines de mètres). Dans ce cas-là, une douzaine de verticales aurait été plus appropriée.

En ce qui concerne le nombre de points par verticale, il ne devrait jamais descendre au-dessous de 3, à l'exception de certains cas où la concentration varie très peu le long de la verticale (forte turbulence, faible profondeur, très faible proportion d'éléments sableux). Une moyenne de 4 points par verticale semble convenir le plus souvent. Cependant, dans le cas déjà cité du Chari à Fort-Lamy, B. Billon a constaté dans les trois derniers mètres au-dessus du fond une augmentation rapide de la concentration qui nécessiterait de porter de 4 à 6 le nombre de prélèvements par

verticale. L'augmentation de la concentration vers le fond est un phénomène général, mais elle est particulièrement prononcée sur le Chari, alors que souvent elle est largement compensée par la diminution de la vitesse et ainsi affecte peu le calcul du débit solide.

Notons enfin que pour les petits cours d'eau, la densité optimale doit tenir compte des variations du débit solide qui ne doivent pas excéder 10 % pendant la durée des prélèvements.

1.3. Fréquence des prélèvements

La détermination jour par jour des débits solides d'un cours d'eau est assez délicate, car il n'y a pas entre les débits solides et les hauteurs limnimétriques une relation définie, analogue à la courbe d'étalonnage qu'on utilise pour évaluer les débits liquides.

Pour suivre les variations du débit solide, il est donc nécessaire d'effectuer des mesures très fréquentes, à une cadence quotidienne par exemple, pour un cours d'eau dont les variations de débit atteignent couramment 10 % d'un jour à l'autre.

Comme en pratique, il n'est pas possible d'envisager des mesures complètes à une telle cadence, on se contente d'effectuer journellement un, deux ou, tout au plus, trois prélèvements en des points bien définis de la section. Des séries complètes de prélèvements sont effectuées seulement tous les 10 jours. Elles permettent de valoriser les mesures quotidiennes en étudiant la corrélation plus ou moins étroite qui existe entre les résultats fournis respectivement par les mesures simplifiées et par les mesures complètes.

L'imprécision de cette méthode est généralement de l'ordre de 10 % et n'excède pas 20 %. Elle pourra vraisemblablement être réduite en faisant intervenir dans la corrélation certains termes correctifs, comme la hauteur limnimétrique, et en recherchant la position optimale des points de prélèvements journaliers.

Dans le cas des petits cours d'eau sujets à des crues rapides, des prélèvements quotidiens sont insuffisants. Il faut prévoir au moins 3, sinon 5 prélèvements bien échelonnés pendant la montée, le maximum et la baisse de la crue.

Par contre, en saison sèche lorsque le tarissement est très régulier, la cadence peut être hebdomadaire pour les prélèvements simplifiés et mensuelle pour les prélèvements complets.

2. OPERATIONS DE LABORATOIRE

2.1. Décantation et siphonnage

Cette opération peut être effectuée sur le terrain, ce qui évite le transport d'échantillons volumineux, mais pour la précision des résultats il vaut mieux l'effectuer en laboratoire.

Les échantillons transportés dans les jerricans en plastique sont transvasés dans des dames-jeannes de 10 litres étalonnées. Ce transvasement est évidemment inutile si les échantillons ont été directement mis dans des dames-jeannes au moment du prélèvement en rivière. Le remplissage des dames-jeannes a alors été effectué jusqu'à un trait-repère marqué à la base du goulot. Le volume correspondant à ce trait-repère est très voisin de 10 litres et a été mesuré une fois pour toutes avec précision. Grâce à l'étroitesse du goulot, on obtient une erreur sur le volume inférieur à 20 ml, soit une

erreur relative de l'ordre de 2 pour mille. Si un transvasement est nécessaire, l'erreur sur le volume reste très largement inférieure à 1 %, mais il faut opérer avec beaucoup de soin pour éviter toute perte de matière solide qui pourrait adhérer au fond du premier récipient.

Dans chaque dame-jeanne, on ajoute 5 ml d'acide chlorhydrique qui provoque la floculation des particules argileuses et on laisse l'échantillon se décanter pendant au moins vingt-quatre heures. Pour favoriser la décantation, on peut éventuellement chauffer l'échantillon au bain-marie jusqu'à 50 °C, ce qui renforce l'efficacité de l'acide chlorhydrique. Des essais seraient nécessaires pour savoir si un gain appréciable de précision justifie cette opération de chauffage qui permet d'obtenir un liquide parfaitement limpide, mais qui nécessite certaines précautions pour éviter que la dame-jeanne ne se casse (interposer une planchette de bois entre la dame-jeanne et le fond métallique du récipient extérieur).

Après vingt-quatre heures de repos on procède au siphonnage, opération délicate où il faut éviter toute manœuvre maladroite qui remettrait des éléments solides en suspension. On introduit dans la dame-jeanne (de préférence légèrement inclinée au préalable) un tube de siphonnage maintenu de telle façon que son extrémité soit à 3 ou 4 cm du point le plus bas du récipient. La transparence de celui-ci permet de vérifier qu'aucune particule n'a été remise en suspension. S'il n'en est pas ainsi, on attend que la décantation soit à nouveau complète avant de commencer le siphonnage. Vers la fin du siphonnage, il faut également veiller soigneusement à ce qu'aucune particule solide ne soit aspirée.

Le résidu, après siphonnage, est de l'ordre de 300 ml et ne doit pas excéder 500 ml.

2.2. Filtration et dessiccation

On peut opérer directement la dessiccation en transvasant le résidu dans un bécher qu'on place pendant 15 à 20 heures sur une plaque chauffante ou dans une étuve. Cette méthode présenterait un très réel avantage si l'on pouvait se procurer des béchers en verre pyrex présentant, pour des raisons qui apparaîtront en 2.3, les caractéristiques suivantes : couvercle rodé parfaitement étanche, capacité de l'ordre de 600 ml, poids total inférieur à 200 g. Ce type de bécher s'étant révélé introuvable et l'emploi d'un récipient métallique étanche n'étant pas recommandable (l'attaque de l'acide chlorhydrique introduirait des sels parasites), il semble actuellement préférable de procéder à une filtration préalable des résidus de décantation.

Cette filtration présente les avantages suivants :

Possibilité de rincer la dame-jeanne avec un volume d'eau assez important, ce qui évite des pertes de matières solides ;

Elimination des sels dissous, dont le poids peut dans certains cas atteindre une proportion non négligeable du poids des matières solides. On évite ainsi la détermination d'un terme correctif ;

Possibilité de dessécher simultanément dans l'étuve un nombre plus grand de filtres que de béchers et élimination du risque de casse.

La filtration présente cependant des inconvénients par rapport à la dessiccation directe en béchers étanches :

Nécessité de mesurer avant filtration le poids de chaque filtre ;

Temps de filtration assez long, surtout si le *p*H de l'eau est faible.

Absorption rapide d'humidité atmosphérique par le filtre et par le résidu sec dès la sortie de l'étuve, ce qui nuit à la précision des pesées comme on le verra plus loin.

La dessiccation des filtres chargés du résidu de filtration s'effectue dans une étude à 110 °C et doit durer une vingtaine d'heures.

2.3. Pesée

La balance qu'on utilise pour la pesée des résidus secs n'a pas besoin d'avoir une très haute précision. Sur les cours d'eau d'Afrique noire, les concentrations sont le plus souvent comprises entre 25 et 500 g/m^3, ce qui pour des échantillons de 10 litres conduit à des résidus secs variant entre 250 et 5 000 mg. Une balance précise au milligramme est alors largement suffisante. Un trébuchet, dont la précision n'est que de 5 mg, peut même convenir pour la plupart des cas et a fortiori pour certains cours d'eau torrentiels comme les "mayo" du Nord-Cameroun où en crue la concentration dépasse parfois 10 kg/m^3. Cependant, en période d'étiage la concentration peut tomber à des valeurs très faibles : on a ainsi mesuré 2,4 g/m^3 sur la Sanaga à Nachtigall en mars 1969. On peut alors être tenté d'utiliser une balance précise au 1/10 mg, mais cette précision serait assez illusoire compte tenu des diverses manipulations auxquelles sont soumis les échantillons. L'emploi d'une balance à mesure directe présente par contre un net avantage pour réduire la durée des pesées.

Une première précaution à prendre est de ne pas peser les filtres chargés du résidu sec dès leur sortie de l'étuve, mais d'attendre qu'ils se soient refroidis à la température ambiante, ce qui demande environ une demi-heure. Un corps chaud posé sur le plateau d'une balance crée, en effet, des mouvements convectifs de l'air environnant qui conduisent à une erreur par défaut sur le poids. Cette erreur peut atteindre plusieurs dizaines de milligrammes pour un corps porté à plus de 100 °C.

Une seconde précaution indispensable est de laisser refroidir les filtres chargés de résidu sec non pas à l'air libre mais dans un dessiccateur efficace, pour éviter l'absorption d'humidité atmosphérique. Des essais systématiques ont en effet montré que cette absorption est loin d'être négligeable, même dans un climat relativement sec comme celui du Nord-Cameroun. A Maroua, avec une température de 33 °C et une humidité relative de 29 %, le poids des filtres laissés à l'air libre augmentait de 50 à 125 mg (suivant l'importance du résidu sec) pendant les 30 minutes qui suivaient la sortie de l'étuve. A Yaoundé, où le climat est beaucoup plus humide, le gain de poids avec une hygrométrie de 82 % était de 50 mg pour les filtres nus pendant l'intervalle de temps compris entre 15 secondes et 3 minutes après la sortie du dessiccateur. Pour des filtres chargés, le gain de poids dans les mêmes conditions était respectivement de 40, 34 et 43 mg pour des résidus secs de 5 000, 2 000 et 600 mg, soit des augmentations relatives de 0,8, 1,7 et 7,2 %.

Il est intéressant d'utiliser une balance à lecture directe qui permet de réduire à moins d'une minute la durée des pesées. Il est néanmoins indispensable d'effectuer une correction, tout au moins pour la mesure des faibles concentrations.

Si l'on désigne par : P_F, P_R et P_T le poids du filtre, le poids du résidu sec et le poids total à l'instant où l'on sort du dessiccateur le filtre chargé ($t = 0$) ; p_F, p_R et p_T les augmentations de poids dues à l'absorption d'humidité à l'instant de la pesée ($t = 30$ secondes par exemple) ; p'_F l'augmentation de poids du filtre à l'instant de la pesée, s'il était nu, on a évidemment :

$$P_R = (P_T + p_T) - (P_F + p_F + p_R)$$

Dans cette expression de P_R, la valeur de la première parenthèse est fournie avec précision par la balance, mais la valeur de la seconde parenthèse ne peut être connue

que de façon approchée en effectuant avant la filtration une pesée du filtre nu desséché. Cette pesée préalable permet de déterminer la valeur de :

$$P_F + p'_F$$

et fournit une valeur approximative de :

$$P_F + p_F + p_R$$

Pour un climat sec comme celui de Maroua, cette approximation est excellente et conduit à une erreur inférieure au milligramme, si la pensée s'effectue en moins de 3 minutes, ce qui est possible avec un simple trébuchet. Par contre, pour un climat humide comme celui de Yaoundé, on a :

$$p'_F > p_F + p_R$$

Il en résulte pour la détermination de P_R une erreur par défaut qui peut atteindre 15 mg d'après les essais.

Une autre façon d'opérer consisterait à effectuer plusieurs mesures successives de $(P_F + p'_F)$ et de $(P_T + p_T)$ à des intervalles rapprochés (par exemple pour $t = 15, 30, 45, 60$ secondes). L'extrapolation vers l'instant $t = 0$ des courbes de variations de poids en fonction du temps permettrait de déterminer P_F et P_T, les termes p devenant nuls. Cette méthode fournirait un recoupement sinon un gain de précision très sensible, car l'extrapolation resterait assez délicate.

En définitive, l'absorption d'humidité apparaît comme une cause d'erreur importante en climat humide.

La question se pose de savoir s'il est souhaitable de s'affranchir de cette cause d'erreur en essayant d'utiliser à l'avenir des béchers étanches répondant aux caractéristiques mentionnées en 2.2. On se trouverait alors confronté à un nouveau problème, celui de l'évaluation du poids des sels dissous. Sauf le cas d'eaux très douces, il n'est pas sûr que ce problème soit beaucoup plus facile à résoudre que celui posé par l'absorption d'humidité. Ce point demanderait à être précisé.

3. CALCUL DU DEBIT SOLIDE

3.1. Concentration des échantillons

Au terme des opérations de laboratoire, il est facile de déterminer la concentration C (en g/m^3) des échantillons à partir du poids de résidu sec P_R (en mg) et du volume V (en dm^3) des échantillons prélevés :

$$C = P_R / V.$$

L'erreur relative sur le volume V est au plus égale à 5/1000. L'erreur absolue sur le poids P_R peut varier entre 1 et 15 mg environ, ce qui, dans les cas les plus courants (P_R compris entre 250 et 5000 mg), conduit à des erreurs relatives situées entre 2/10000 et 6/100.

L'erreur relative sur la concentration des échantillons varie donc très largement et peut s'échelonner entre 5/1 000 et 6,5/100, hormis le cas des eaux extrêmement peu chargées des écoulements d'étiage, pour lesquelles l'erreur relative peut être de l'ordre de 100 %.

3.2. Méthode de la double interprétation graphique

Le calcul des concentrations C des échantillons prélevés sur une verticale et la mesure des vitesses V effectuée au moulinet sur cette même verticale permettent de tracer les courbes de variations de C et de V en fonction de la profondeur y, puis d'en déduire la courbe de variations de la densité de débit solide $d = C \times V$ en fonction de y. Le tracé préalable des deux courbes $C(y)$ et $V(y)$ permet de tenir compte dans le tracé de $d(y)$ de toutes les mesures effectuées sur la verticale dans le cas fréquent où les mesures de vitesse sont plus nombreuses que les mesures de concentration.

La vitesse étant un facteur du débit solide aussi important que la concentration, il est également intéressant de tenir compte des verticales le long desquelles ont été effectuées uniquement des mesures de vitesse. En traçant, comme l'a suggéré J. Colombani, les courbes de variations de C et V en fonction de la distance horizontale x, pour différentes valeurs de y (par exemple : $y = 0{,}5\ m,\ 1\ m,\ 2\ m,\ 3\ m$, etc.), il est possible de déterminer les courbes correspondantes $d(x)$, puis d'interpoler les courbes $d(y)$ correspondant aux verticales pour lesquelles on dispose uniquement de mesures de vitesse.

Le calcul du débit solide s'effectue ensuite par une double intégration graphique, de la même façon que le calcul classique du débit liquide. Le planimétrage des surfaces délimitées par les courbes $d(y)$ correspondant aux diverses verticales donne la valeur des débits solides m par unité de largeur. Le planimétrage de la surface délimitée par la courbe $m(x)$ donne la valeur du débit solide en suspension M (en grammes par seconde).

Comme pour le calcul des débits liquides, on peut effectuer la double intégration graphique d'une autre façon. On trace les courbes d'égale densité de débit solide dans la section de mesure. On planimètre les surfaces cumulées délimitées par ces courbes pour des valeurs décroissantes de d et on en déduit une courbe $S(d)$ donnant la surface de la partie de la section où la densité de débit solide est supérieure ou égale à une valeur donnée. Le planimétrage de la surface délimitée par cette nouvelle courbe donne la valeur du débit solide en suspension M.

3.3. Méthode simplifiée

La méthode précédente revient à déterminer le débit solide M par une intégrale double qu'on peut écrire sous la forme générale :

$$M = \iint_S C \,.\, V \,.\, \mathrm{d}S$$

En désigant par Q le débit liquide dans la section de mesure, on peut définir une concentration moyenne C_p telle que :

$$M = C_p \, x \iint_S V \,.\, \mathrm{d}S = C_p \,.\, Q$$

C_p est la moyenne des concentrations pondérées par la vitesse. Une méthode de calcul simplifiée consiste à prendre comme valeur approchée de C_p la moyenne arithmétique C_a des concentrations des échantillons et d'en déduire le débit solide par la relation :

$$M = C_a \,.\, Q$$

Les concentrations étant généralement plus fortes au voisinage du fond où les vitesses sont au contraire plus faibles, on doit s'attendre à ce que C_a soit légèrement supérieure à C_p. C'est ce qu'ont confirmé les essais de J. Colombani, tout au moins dans le cas où des difficultés pratiques n'avaient pas empêché d'effectuer des prélèvements près du fond.

Lorsque les points de prélèvement sont bien répartis dans la section de mesure et que les concentrations ne varient pas très fortement d'un point à l'autre de la section, la méthode simplifiée conduit à une surestimation du débit solide de l'ordre de 2 à 3 % en moyenne par rapport à la méthode complète. Par contre, dans le cas de concentrations très variables dans la section, l'écart serait peut-être de 10 %.

La méthode simplifiée présente l'avantage pratique de permettre le calcul des débits solides sans avoir à effectuer de mesures de vitesse, à condition que l'étalonnage des débits liquides soit déjà bien connu. Il faut, bien entendu, veiller à ce que la méthode soit applicable sans perte de précision excessive et apporter éventuellement un terme correctif par comparaison avec des mesures complètes.

La simplification de la méthode peut encore être poussée plus loin en réduisant les prélèvements d'échantillons à un tout petit nombre (entre 1 et 3). Comme déjà indiqué en 1.3, cette méthode présente le gros avantage de permettre une évaluation quotidienne du débit solide. Pour que la précision reste acceptable, il est cependant indispensable qu'une corrélation satisfaisante soit établie entre la concentration moyenne C_p déterminée par des mesures complètes et la concentration moyenne C_a déterminée à partir d'un nombre très réduit d'échantillons.

4. PRECISION DES RESULTATS

Nous avons vu que la concentration des échantillons peut être déterminée dans la quasi-totalité des cas avec une erreur relative comprise entre 5 ‰ et 6,5 %, suivant la turbidité des eaux.

Dans le calcul du débit solide interviennent, en outre, les erreurs d'échantillonnage des concentrations et des vitesses. Ces erreurs sont dues aux imperfections des appareils d'échantillonnage ainsi qu'à la densité et à la durée limitées des échantillons.

Dans l'état actuel des recherches de l'ORSTOM, il n'est pas possible d'évaluer avec beaucoup de rigueur ces différentes causes d'erreur. Nous pensons néanmoins que dans la plupart des cas où la teneur des eaux en sables est très minime, l'erreur globale sur le débit solide en suspension reste comprise entre 5 et 10 %, à la condition expresse qu'on prenne toutes les précautions que nous avons recommandées, tout particulièrement en ce qui concerne les opérations de laboratoire. Ce n'est que dans le cas d'eaux très peu chargées (moins de 25 g/m^3) que l'erreur relative sur le débit solide peut dépasser largement 10 %, surtout si les opérations de pesée sont effectuées dans une ambiance très humide. L'absorption d'humidité par les résidus secs peut alors devenir, si l'on n'y prend pas garde, une cause d'erreur très importante.

Pour terminer, nous dirons quelques mots des mesures granulométriques qui sont classiques mais qui présentent des difficultés à cause de la faible quantité de sédiments recueillis. Il est nécessaire d'opérer sur plusieurs grammes de matière sèche, ce qui oblige généralement à grouper les résidus secs de plusieurs échantillons. On groupe, dans la mesure du possible, des échantillons prélevés à des profondeurs identiques. Pour les argiles et limons (diamètre inférieur à 0,05 mm), qui sont

largement prédominants dans la plupart des cas, on détermine la courbe granulométrique par mesure de la vitesse de sédimentation dans une colonne d'eau immobile. Pour les sables on opère en principe par tamisage, mais les quantités recueillies sont généralement trop faibles pour donner des résultats très significatifs.

BIBLIOGRAPHIE / BIBLIOGRAPHY

Touchebeuf de Lussigny, P. Etude des transports solides en Afrique noire et à Madagascar. Annuaire hydrologique de l'ORSTOM. 1958.

Colombani, J. Contribution à la méthodologie des mesures systématiques de débits solides en suspension. Cahiers ORSTOM, série Hydrologie, vol. IV, n° 2, 1967.

Billon, B. Mise au point des mesures de débits solides en suspension (République du Tchad). Cahiers ORSTOM, série Hydrologie, vol. V, n° 2, 1968.

Nouvelot J. F. Mesure et études des transports solides en suspension au Cameroun. Cahiers ORSTOM, série Hydrologie, vol. VI, n° 4, 1969.

VII Measurement of water quality

Mesure de la qualité de l'eau

Instrumentation considerations for studies of quality of runoff from small agricultural watersheds

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Abstract. Evaluating water quality on small agricultural basins is directed toward relating pollution to manageable or predictable factors. Continuous monitoring and telemetering is limited to a few parameters and to continuously flowing streams. Discrete samples must be collected for subsequent analysis when surface runoff from individually treated fields and intermittent flowing watersheds is to be studied.

The sampling programme must be determined by the specific goals of research. Sample size, frequency, storage, preparation for analysis, analysis, and interpretation must be carefully planned to ensure interpretable results. Devices that automatically take several samples, refrigerate them, and indicate on a hydrograph the exact time and, therefore, flow rate at sampling times are described. Merits of various flow sensors, intakes, distribution systems, containers, and event recorders are discussed.

CONSIDERATIONS INSTRUMENTALES POUR L'ETUDE DE LA QUALITE DE L'EAU DANS DE PETITS BASSINS AGRICOLES

Résumé. Les mesures de la qualité de l'eau dans de petits bassins agricoles visent à relier la pollution à des facteurs maniables ou prévisibles.

Le contrôle et la télémétrie continus sont limités à quelques paramètres et à des cours d'eau permanents. On doit rassembler des échantillons séparés pour analyse ultérieure lorsque le ruissellement provient de champs examinés individuellement ou de bassins d'écoulement intermittents. Le programme de prélèvements doit être déterminé par les buts spécifiques de la recherche. La dimension des prélèvements, leur fréquence, leur entreposage, leur préparation pour l'analyse, leur analyse et l'interprétation de celle-ci doivent être conçus avec soin pour donner des résultats susceptibles d'être interprétés. L'auteur fournit la description d'un appareil qui prend automatiquement plusieurs prélèvements, les refroidit et indique sur un hydrographe l'heure précise et, par conséquent, le débit au moment du prélèvement. Les mérites respectifs de différents systèmes de mesure de débits, de prélèvement, de distribution, de stockage et d'enregistrement sont passés en revue.

CONSIDERACIONES SOBRE EL EQUIPO PARA ESTUDIOS DE CALIDAD DEL AGUA EN PEQUEÑAS CUENCAS AGRÍCOLAS

Resumen. La evaluación de la calidad del agua en pequeñas cuencas agrícolas va dirigida a relacionar la polución con factores manejables o predecibles. Los monitores y telemetría continua se limitan a unos pocos parámetros y a corrientes en flujo continuo. Se deben recoger muestras discretas para el subsiguiente análisis cuando se va a estudiar la escorrentía superficial de los campos y cuencas de corrientes intermitentes.

El programa de toma de muestras debe estar determinado para metas específicas de investigación. Se deben planificar cuidadosamente el tamaño de la muestra, la frecuencia, el almacenamiento, la preparación para análisis, el análisis y su interpretación, a fin de asegurar resultados interpretables. Se describen los dispositivos que automáticamente toman varias muestras, las refrigeran e indican en un hidrograma el tiempo exacto, y por tanto el valor del caudal y los tiempos de toma de muestras y se discuten las ventajas de los diversos detectores de flujo, tomas, sistemas de distribución, depósitos y registradores.

СООБРАЖЕНИЯ ОТНОСИТЕЛЬНО ОБОРУДОВАНИЯ ДЛЯ ИССЛЕДОВАНИЯ КАЧЕСТВА СТОКА С МАЛЫХ СЕЛЬСКОХОЗЯЙСТВЕННЫХ ВОДОСБОРОВ

Резюме. Оценка качества воды на малых сельскохозяйственных бассейнах имеет цель найти связь между загрязнением и факторами, с помощью которых можно управлять им или предсказать его. Непрерывное контролирование и телеизмерение ограничено несколькими параметрами и реками, имеющими непрерывный сток. Отдельные пробы должны быть взяты для последующего анализа, когда изучается поверхностный сток и индивидуально обработанных полей и водосборов с прерывающимся стоком.

Программа взятия проб должна определяться обосыми целями исследования. Размер проб, частота, хранение, подготовка к анализу, анализ и интерпретация результатов должны быть тщательно запланированы для обеспечения объяснимых результатов.

Устройства, которые автоматически берут несколько прос, охлаждают их и отмечают на гидрографе точное время, и таким образом фиксируется величина расхода во время взятия проб. Рассмотрены преимущества различных датчиков стока, водоприемников, систем распределения, контейнеров и регистраторов событий.

For many years, hydrologic research in agriculture has been directed towards gaining an understanding of the factors controlling the volume and rate of runoff and erosion from farmland. Today, another facet is added to this research effort, caused by man's activities that affect the quality of this water. It is obvious that industrial and city wastes degrade water quality. However, criticism is also directed towards agriculture for adding to the problem. But its contribution is not so obvious; one cannot see chemicals which were applied to the land in the muddy water flowing from cropland. People are alarmed at the polluted river and lake waters rendered unfit for human, industrial, or agricultural use, polluted by human, industrial, and agricultural activities. The public is demanding programmes to reduce pollution of water. But some programmes must be based on research findings on pollutant transport related to basin hydrology.

This paper reports experience gained in developing instrumentation needed in research on movement of agricultural chemicals and waste from farmland into water bodies. Studies were made on farmland of the North Appalachian Experimental Watershed, Coshocton, Ohio, under natural farming conditions. Here, the effects of historic and present treatment could be identified, since background observation on the basin land use and hydrology had been made for over twenty-five years. Under these conditions, the research goal was set to evaluate the present mechanism of pollutant transport and to develop new practices to minimize pollution and maximize water quality of farmland runoff.

A straightforward approach to achieving the research goal involved applying different chemical and land management practices to selected basins and analysing samples of subsequent runoff. In 1966, such a programme was started at the Coshocton, Ohio, station. During the period 1966-69, eight 1-ha basins have been instrumented with automatic runoff samplers.

Most of the land in these experimental basins has been farmed in a four-year corn, wheat, meadow, meadow rotation—a common practice in this region. Ploughing to 18 cm followed by disking and harrowing, either on the contour or straight across the slope, is the normal seedbed preparation for corn. Weeds are controlled by a

combination of herbicides and cultivation. One more disking after corn harvest prepares the soil for wheat planting. Grass-legume seeding is made in the wheat period.

These tillage operations, combined with moderate fertilizer applications, determine the surface conditions from which the sampled runoff comes. The rate, volume, sediment load, and chemical content of the runoff are strongly influenced by these operations and differ widely during the four-year rotation. Any sampler, regardless of the tests performed on the samples, must, therefore, be able to deliver representative portions of runoff varying from large to small volume, running off at a fast or slow rate, and carrying strongly or weakly polluted loads.

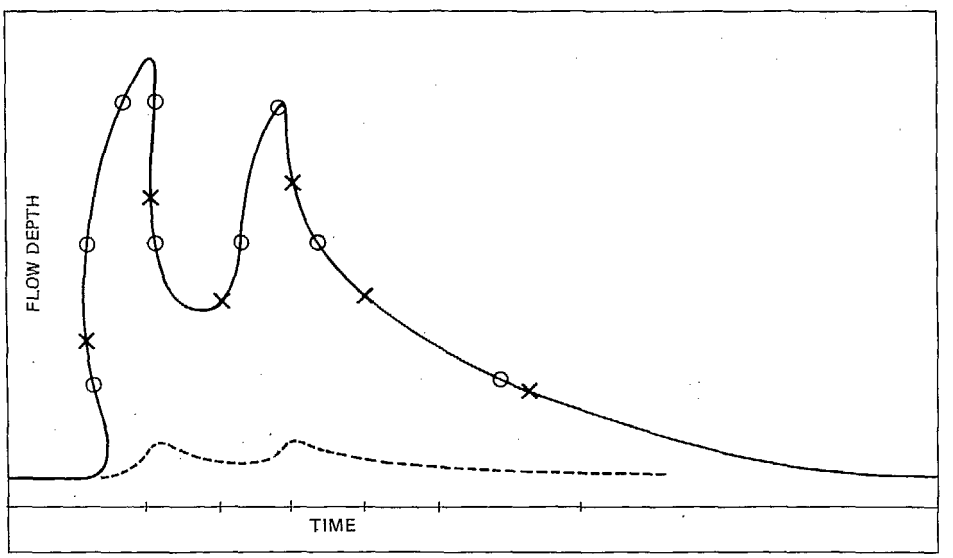


Figure 1 – *Sampling pattern when interval is determined by stage (O) timer (x), or continuous proportionate sampler (. . .), with event marker (") indicating timer-determined samples*

These factors describe conditions under which the sampling system must perform. The chemist and the analyst set forth other specifications the sampler must meet. They are presented below.

SAMPLE CHARACTERISTICS

General. Samples are taken for specific reasons—to describe certain characteristics of runoff. Specific research objectives define the characteristics that each sample must have in order to best satisfy the demands of the analytical and interpretative processes. Therefore, the nature of the samples to be taken can be determined only after the exact methods of analysis and interpretation have been determined. Some important sample characteristics are discussed below.

Size. The volume of each sample is determined by the number of analyses performed and by the amount of sample needed for each analysis. In one case, 250 ml samples were adequate for simple pH and electrical conductivity analysis in evaluating the

effect of surface mining for coal on the quality of surface and subsurface water flowing therefrom. At the other extreme of complexity, the Federal Water Pollution Control Administration, United States Department of the Interior, lists 47 parameters for setting individual standards for water quality (FWPCA, 1969). One of these parameters is pesticides which, for analytical procedures, has been divided into 10 different chlorinated hydrocarbon and 13 organic-phosphorus insecticides. A sample of 250 l may be required to evaluate all of these parameters.

For studies requiring separate analysis of liquids and solids, the sample must be large enough to provide enough solid for the required analyses as well as a subsample for determining sediment concentration.

Physical and chemical properties. For some studies, the liquid and solid phases of runoff are important, either separately or combined. The study objectives may require physical and chemical evaluation of the solids in runoff as carriers of adsorbed potential pollutants. In this case, the exchange of ions between the liquid and solid phases during the interim between sampling and laboratory analysis becomes important. Also the possibility of biological activity in the sample altering the liquid phase-solid phase equilibrium suggests the need for refrigerating or freezing some samples immediately. Separating the liquid and solid phases by centrifugation at a given time after sampling standardizes the period of ion interchange.

Representativeness. Samples may be taken continuously or intermittently as shown in Figure 1. During steady flow, intermittent sampling may describe the quality of flow between samples adequately. However, extrapolating water quality data across hydrograph peaks from samples taken before and after the peaks may give erroneous results. Under these conditions, a continuous proportionate sampler may describe the total flow more adequately.

SAMPLER CHARACTERISTICS

Just as the research objectives define the characteristics of the samples, required sample characteristics define the nature of the sampling equipment.

Samples must be representative of the total runoff, storm by storm. The sampler must operate automatically, remaining on a standby basis between storms, ready to begin sampling at the onset of runoff. Few human samplers can meet this requirement.

Automatic intermittent samplers are equipped with a *flow sensing switch* to start and stop the sampler. One very satisfactory method uses a styrofoam float to tip a mercury switch (Harrold *et al.*, 1967) or trip a micro-switch (Miller *et al.*, 1969). The float may be set to activate the sampler circuit when the runoff reaches a predetermined level in the stilling well of a runoff measuring flume or weir. Lag in the switch linkage may be adjusted by trial in the field to give the desired relationship between on and off levels, or two switches may be used–the "on" switch at one level and the "off" switch at another. The electrolytic property of the rising water level has been used to instigate sampling with two electrodes positioned at the desired "turn-on" height (FWPCA, 1967). The pressure sensing switch (Harrold *et al.*, 1967) does not consistently turn the system "on" and "off" at the desired levels.

The flow sensing switch may also be used to programme sample operation at specific flow rates or flow depths (Fig. 2). As the float wheel rotates with changes in

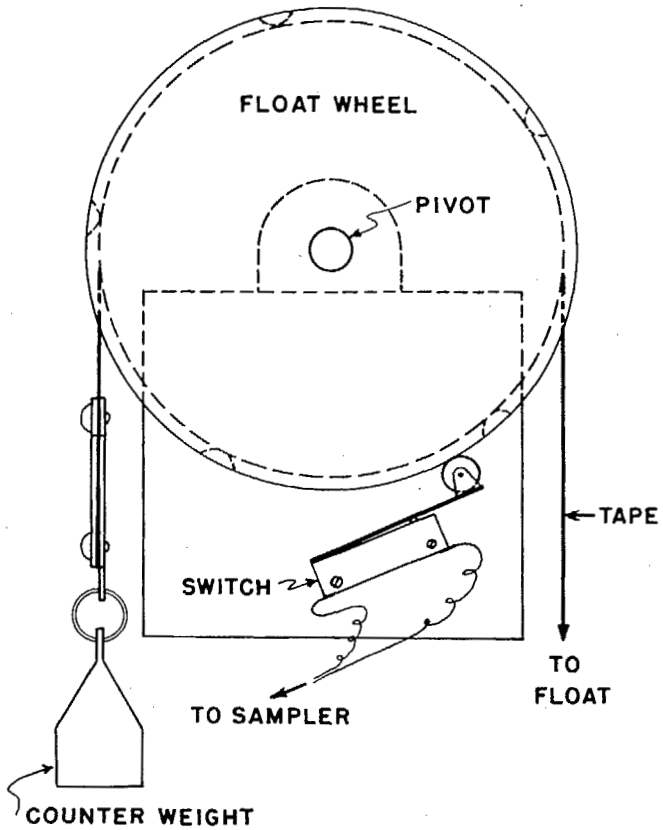


Figure 2 – *Delta stage sensor*

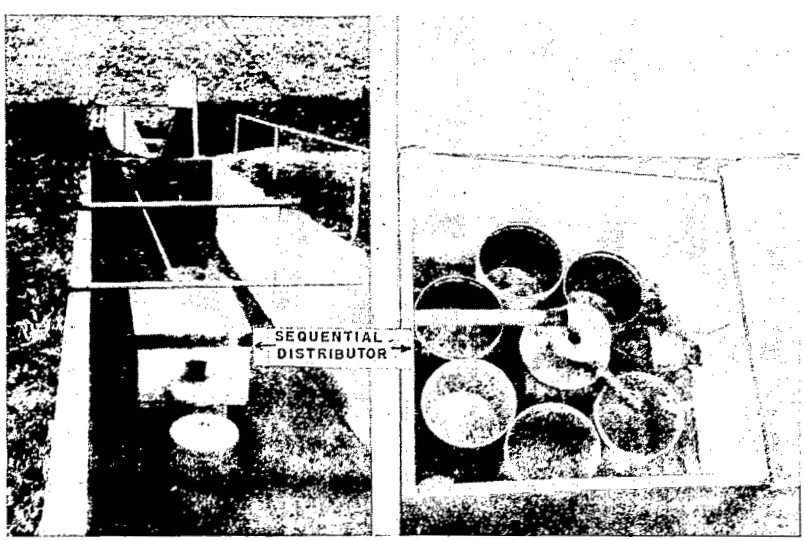


Figure 3 – *Gravity-powered sequential sampler with continuous proportionate intake*

flow depth, the rider wheel on the switch falls into a notch, the switch closes and a sample is taken.

When the general shape of the hydrograph can be predicted, a timer system may be used to *pre-programme the sampling sequence* (FIASP, 1969 ; Harrold *et al.*, 1967 ; Miller *et al.*, 1969). The sampling interval may be adjusted for the expected runoff or may be determined by stage sensing switches (FIASP, 1969).

In contrast, a *continuous proportionate sampler* may better meet certain study objectives. The Coshocton Wheel (Parsons, 1954) delivers a continuous sample of 0.001 of the runoff rate into a container. Some studies that require samples of segments of storm runoff use a sequential distribution system (Fig. 3). In this case sampling is continuous, resulting in discrete samples as determined by runoff volume and container size.

Intake systems vary to be compatible with flow rates, sediment loads, and site restrictions. In turbulent or sediment-free flow, a single pipe-end intake may suffice (FIASP, 1969 ; Miller *et al.*, 1969). Overfall from a runoff measuring flume into a turbulence box ensures a representative sample from a single point pick-up. A screen wire trash guard may be needed to protect pumps and valves, but its effect upon the sample must be considered.

In laminar flow at the entrance of a rate measuring flume, a three-point pick-up (one near the water surface, one near the flume floor, and the third midway between the other two) has proved satisfactory (Edwards *et al.*, 1969 ; Harrold *et al.*, 1967). A hollow intake wand having a float on the downstream end and pivoted at the flume floor at the upstream end, automatically positions the three intake ports at the desired sample integrating levels. However, large trash or sediment loads tend to clog the intake ports.

Delivery and distribution of the samples to the containers may be effected by pumps and valves, vacuum, or gravity. At Coshocton, individual solenoid valves pass sample material to each of 12 jugs in a refrigerator (Edwards *et al.*, 1969 ; Harrold *et al.*, 1967). The Chickasha sampler (Miller *et al.*, 1969) distributes the flow to a revolving bottle rack without passing it through a valve, eliminating the possibility of sediment clogging the valves which might ruin the entire sampling sequence. A pre-applied vacuum in each sample container allows atmospheric pressure to fill jars without valves in the sampler used by the Miami Conservancy District (Gilbertson, 1967). The Coshocton large volume sampler (Bentz and Edwards, in press) uses neither pump nor valves; gravity delivers the sampled flow through 5 cm diameter pipes.

The sample distribution system may be powered by alternating line current (110 or 220 volt), direct current battery (6-or 12-volt wet cell, rechargeable; or 6-volt dry cell, nonrechargeable), or gravity.

An *event marker* to indicate when each sample is taken enables the concentration of water quality parameters in discrete samples to be related to flow rates and volumes. The indication may be a pip on the hydrograph (Miller *et al.*, 1969) or a tick on the margin of the chart paper (Edwards *et al.*, 1969) as shown on Figure 1.

SUMMARY

Instrumentation for water quality research is determined by characteristics that samples must have to best answer the research objectives. Desired sample characteristics determine sampler characteristics. The sampler is a composite of the best

adapted flow sensor, power source, intake, distribution system, delivery system, container, and event recording device.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Bentz, W. W.; Edwards, W. M. In press. Coshocton large volume sampler. *U.S. Dept. Agr. ARS.*

Edwards, W. M.; Bentz, W. W.; Harrold, L. L. 1969. Improvements in automatic sampling equipment used to determine extent of pollution in runoff from agricultural watersheds. *U.S. Dept. Agr. ARS 41-151,* 16 p.

Federal Inter-Agency Sedimentation Project, FIASP. 1969. Operating and Servicing Manual: PS-67 Pumping Sampler. St. Anthony Falls Hydraulic Laboratory, Minneapolis, Minn. 23 p.

Federal Water Pollution Control Administration, FWPCA. 1967. Installation, operation, and maintenance of LF-1 and LF-2 Organics Samplers for water, U.S. Dept. Interior, 12 p.

——. 1969. Orientation to automated instrumentation in surveillance. U.S. Dept. Interior, Administration Training Manual.

Gilbertson, J. G. 1967. Serco automatic sampler–1/2 gal. refrig. Sanford Products Corp, Minneapolis, Minn. 5 p.

Harrold, L. L.; Barrows, H. L.; Bentz, W. W. 1967. Automatic sampling technique to determine extent of pollution in runoff from agricultural watersheds. *U.S. Dept. Agr. ARS-41-136, 12 p.*

Miller, G. E.; Allen, P. B.; Welch, N. H.; Rhoades, E. D. 1969. The Chickasha sediment sampler. *U.S. Dept. Agr. ARS-41-150,* 14 p.

Parsons, D. A. 1954. Coshocton-type runoff sampler. *U.S. Dept. Agr. TP 124,* 16 p.

Determination of the BOD_5 in running waters by means of biological water analysis

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Abstract. The discharge of organic oxygen-consuming substances into our rivers and canals is still one of the main problems to be faced in keeping these waters clear. The method of determining the BOD_5 content from the biological state allows the amount of pollution to be determined.

In controlling the waters of Rheinland-Pfalz, the usefulness of this method has been proved repeatedly. In some cases it was found that the quantity of pollution in a river or canal, measured as BOD_5 content, could only be decided from determination of the biological state, and these measurements were employed for management decisions. Examples are quoted from the practice of the Rheinland-Pfalz water supply system.

DETERMINATION DE LA DBO_5 DES EAUX COURANTES AU MOYEN D'ANALYSE BIOLOGIQUE

Résumé. La charge de nos canaux de drainage en substances organiques consommant de l'oxygène représente toujours le problème principal sur lequel portent nos efforts en vue de préserver la propreté des eaux. La mesure continue de la demande biologique d'oxygène (DBO_5) comme meilleur moyen d'analyse pour déterminer cette pollution n'est jusqu'ici pratiquement possible que par conversion après détermination de l'état biologique (Klotter, Hantge 1966).

Lors de la surveillance des eaux en Rheinland-Pfalz, l'utilité de cette méthode a été démontrée à maintes reprises ; dans d'autres cas, il s'est même révélé que la charge d'un canal de drainage, mesurée à partir de la demande biologique d'oxygène, ne peut être obtenue que par conversion de l'état biologique et c'est ensuite seulement qu'il est possible de prendre des décisions importantes concernant l'aménagement des eaux. L'auteur cite des exemples tirés de l'expérience acquise en matière d'alimentation en eau en Rheinland-Pfalz.

DETERMINACIÓN DEL DBO_5 EN AGUAS SUPERFICIALES POR MEDIO DE ANÁLISIS BIOLÓGICO DEL AGUA

Resumen. La existencia en los cauces de drenaje de sustancias orgánicas consumidoras de oxígeno es todavía uno de los principales problemas en el propósito de mantener las aguas limpias. La medición corriente del DBO_5 como análisis más útil para determinar esta contaminación ha sido posible a este fin prácticamente sólo por conversión, después de determinar el estado biológico (Klotter, Hantge 1966).

Al controlar las aguas de Rheinland-Pfalz, se ha probado en repetidas ocasiones la utilidad de este método. En otros casos se encontró que la carga de un canal de drenaje, medida por el DBO_5, se puede obtener sólo por análisis del estado biológico, y que solamente después es posible tomar importantes decisiones. Se citan ejemplos de la práctica en el sistema de suministro de agua de Rheinland-Pfalz.

ОПРЕДЕЛЕНИЕ BOD_5 В ТЕКУЩЕЙ ВОДЕ С ПОМОЩЬЮ БИОЛОГИЧЕСКОГО АНАЛИЗА

Резюме. Загрязнение дренажных каналов органическими, содержащими кислород, веществами по-прежнему является одной из главных проблем в наших стремлениях содержать довы чистыми.

Текущее измерение BOD_5, как наиболее полезный анализ для определения этого водозагрязнения, до сих пор было практически возможно только путем конверсии после определения биологического состояния (Клотер, Хантге, 1966).

При контроле качества вод земли Рейн-Пфальц полезность этого метода доказана неоднократно.

В других случаях было установлено, что насыщение дренажных каналов, измеренное как BOD_5, может быть получено только путем конверсии биологического состояния, после чего было бы дозможно принять важные гидрологические решения.

Примеры взяты из действия системы водоснабжения земли Рейн-Пфальц.

Water resources agencies consider it one of their main targets to keep the proportions of oxygen in water within such limits as to always ensure the continuation of aerobic processes. We know that water without oxygen cannot be used for drinking and utility supply without prior treatment, for example the purification of ground water includes treatment by aeration. If whole lakes become short of oxygen it is very difficult to bring about a reversal, since the lack of oxygen entails quite a series of reduction processes. In addition, the amount of oxygen needed for a return to the aerobic phase is so great that re-aeration by mechanical means is impractical in most cases.

At present many of our European lakes are in an anaerobic condition and intense efforts are required even to decelerate the process. For instance, if it were possible to prevent any waste water getting into the lake, such as by the construction of a ring of conduits, considerable amounts of nitrogen and phosphorus compounds would continue to get into the lake with the ground and drainage waters. This would continue the anaerobic processes by secondary pollution after the production of plankton. The nutritive substances supplied to the lake would remain there in a closed cycle, so that the low oxygen content would become an almost permanent feature.

The condition of our running waters can be considered more favourable since, unlike lakes there is normally no accumulation of decaying matter because it is carried away by the current. Similarly, conditions are far more favourable for the re-aeration of river water. If we charged our lakes with the same quantities of organic and nutritive substances as in the Rhine, for instance, they would have been without oxygen for a long time.

In spite of the increasing number of purification plants incorporating biological treatment, all of them serving to improve the oxygen conditions in our rivers and canals, their oxygen supply, taken as a whole, will continue to deteriorate. The heavy precipitation from 1965 to 1968, which brought increased river flows, obscured the fact that there was also an increase in the oxygen-consuming substances flowing into the rivers. But when river flows decreased in the summer and autumn of 1969, the increasing deterioration in the supplies of oxygen was plainly revealed. The number of dying fish the Regional Office of Hydrology had to deal with in 1969 was three times that for 1968. In nine cases out of a total of forty-two the cause of death could not be determined because the samples were taken too late. Of the remaining cases of dying fish, 39 per cent were caused by an inflow of chemicals and 61 per cent by organic, oxygen-consuming substances. In these cases, the lack or absence of oxygen played a decisive role.

Another example of the increase of oxygen-consuming substances in our rivers comes from the Rhine. The Regional Office of Hydrology carried out determinations of the oxygen content in the Rhine between Wörth (opposite Karlsruhe) and Ludwigshafen in 1964, a drought year. During October of that year, values in the range 7.4 to 8.4 mg/l were recorded, whereas in July these values were between 6.8 and 7.9 mg/l. Measurements carried out in 1969, sometimes under unfavourable weather conditions (particularly fog) and a similar flow of water, gave records of oxygen values in the range 3.8 to 5.2 mg/l. Concentrations in October 1969 were, on average, 2.5 mg/l lower than those for 1964, corresponding to 83 per cent and 45 per cent of the saturation values respectively (7.1 versus 4.6 mg/l). From the conditions downstream of Mannheim-Ludwigshafen it could be computed that upstream of Karlsruhe the inflow of organic substances corresponds to an amount of contamination of 7-10 million inhabitants equivalent value 40 (EGW_{40}). In 1969 this resulted in a considerable oxygen deficiency in the Ludwigshafen-Mannheim region, this deficiency being further increased downstream of this location by the influx of organic, oxygen-consuming substances. Even if in the near future the greater part of the pollution were to be prevented by the purification plants which are under construction, it is our duty to continue to observe closely the state of the oxygen balance and the influx of further oxygen-consuming substances, particularly because of the construction of nuclear power stations. We find similar conditions in many other rivers and drainage channels. Hence it is most important to obtain the necessary information on the inflow of oxygen-consuming substances now and in the future.

As this sampling of rivers and channels needs to be carried out at a large number of sites, it is fortunate that the simple and rapid determination of the BOD_5 content is possible by determination of the biological state of the water samples. For this purpose we use the curve determined by the Regional Office of Hydrology (Klotter and Hantge, 1966) which makes it possible to use the biological state value to assess the BOD_5 value. The existence of a close correlation between the biological state and the BOD_5 value has also been demonstrated by other authors (Reimann, 1966; Tümpling, 1967; Sladecek, 1965). With the aid of examples it is intended to show that, by using this relationship, an assessment can be made of the influx of organic, oxygen-consuming substances and thus on the oxygen balance of running waters.

A small brewery discharged a quantity of pollution of approximately 23 000 EGW_{40} into a small canal prior to the construction of a purification plant. When the flow in the canal was above normal, the inflow of organic substances produced favourable nutritive conditions for the brook trout, but fish died frequently during drought periods because of lack of oxygen. Eventually legal proceedings were instituted against the brewery. The water authority then ordered the brewery not to discharge the bulk of the residue of the hops, draff, wort vat barm and sediments. Chemical analyses by the Regional Office of Hydrology showed that by these measures the amount of pollution could be reduced from 24 000 to 2 080 EGW_{40}. In this way the contamination load of the canal was reduced by 91 per cent, producing the same effect as a purification plant with a biological section.

This result was viewed sceptically by several parties as the anti-pollution measures were expensive and there was no control over them. Hence, it was necessary to monitor the biological state upstream and downstream of the brewery. The upstream value was found to be $z = 1.5$, while 150 m downstream of the discharge, the value of z was 3.4. From the curve these figures correspond to BOD_5 values of 2.8 and 12 mg/l respectively. The average flow of water of the canal was about 95 l/sec. Thus the BOD_5 load on the upstream side of the point of discharge amounted to 23 kg

BOD_5/day, but downstream of the discharge it was 98 kg BOD_5/day. This difference of 75 kg BOD_5, or 1 880 EGW_{40}, is the amount of pollution daily entering the canal, confirming the results of the chemical analysis.

Pollution by paper-mills has developed into a serious problem on a number of canals in the Federal Republic of Germany. Filtration prevents paper sludge from getting into the water, but it is known, however, that considerable amounts of oxygen-consuming substances get into the waste water. For cellulose and waste paper this amounts to 2-19 kg BOD_5/ton, and for mechanical wood pulp to 0.4 kg BOD_5/ton. Since the water consumption of paper-mills is rather high, the discharge of organic substances by the paper-mills has been hardly noticed hitherto because of the low BOD_5 in the waste water (20 – 30 mg/l). Paper and cardboard production has more than trebled during the last 20 years and will continue to increase, so that many of our rivers and canals have developed problems which cannot be solved by chemical analysis alone.

In one case, a cardboard mill manufactures 220 tons of cardboard per day. Waste water analyses and the amount of waste water indicate that the quantity of pollution amounts to about 480 kg BOD_5, or 12,000 EGW_{40}, and this is discharged into a brook. The effects on the brook are obviously serious. Determination of the biological state values upstream and downstream of the waste water discharge, and the calculation of the BOD_5 values with the total flow of water for the preceding four weeks produced an estimate of the quantity of pollution of 23,000 EGW_{40}. This agrees with the results of the chemical analysis of the brook water. The differences in the quantity of pollution, namely 12 000 compared with 23 000 EGW_{40}, can be explained by the fact that the starch added during production causes a considerable *Sphaerotilus* growth in the brook. The final products of metabolism and the decaying cells cause an additional contamination of the brook of more than 90 per cent of the amount of organic substances as determined by chemical analyses of the waste water. Thus only the biological analysis method of assessing the BOD_5 content could prove the real pollution load of the brook.

One of the most important components of a basic hydrological network is the monitoring and regulation of cleanliness. First of all the regulations for cleanliness provide a standard for measuring the present condition of rivers and channels and these standards specify the measures that have to be taken in order to reach them. In the case of the standards for larger rivers a tremendous number of analyses would be required to establish the BOD_5 content by conventional methods. Furthermore, there would be inaccuracies in polluted stretches due to the intermittent discharging of waste water. In our experience, data of sufficient accuracy can be obtained rapidly from the biological state curve. Moreover, the main aim of uniform cleanliness regulations is to maintain a uniform α/β-mesosaprobic condition throughout the river. The average BOD_5 value that can be expected in the river or canal can be computed.

There is also self-purification of water due to the natural decomposition of oxygen-consuming substances as a function of time. It was found, when the standards were being drawn up, that these decomposition rates are unique for each river or canal. Self-purification curves can be determined in a simple manner by biological analyses and BOD_5 conversion, if the BOD_5 is determined at a number of points in the river downstream of the waste water influx. Thus this relationship between the biological state and the BOD_5 content can be used in many fields of hydrology. One should realize, however, that although knowledge of the BOD_5 content will reveal to what extent oxygen is consumed, a large number of chemical and biological analyses are required for a detailed evaluation of water quality.

However, as was shown by the much-quoted dying fish in the Rhine in 1969, we are today hardly in a position to evaluate accurately the water quality of the Rhine with regard to its treatment and its suitability for drinking water. Who had known before these fish deaths that pesticides of various origins, even in very small quantities, have always been found in the Rhine? Who can say what substances will be passed into the Rhine during the next year?

The analysis of these and similar substances is so difficult and expensive that, in the Federal Republic of Germany, there are a very few places where such determinations can be carried out. In this respect it must be taken into consideration that the effect on the environment of such infinitesimally small quantities of substances may only be discovered after a number of years. For example DDT, which was synthesized in 1872 and shown to be an insecticide in 1939, was being produced in the forties by the thousands of tons in America.

For some years now we have known that chlorinated hydrocarbons (not only DDT) are stored in the bodies of warm-blooded animals and are not broken down. Therefore, it is impossible to fix a limit of harmfulness; practically any amount getting into the digestive tract, for instance, together with food or drinking water, is stored and has led to damage, e.g. of whole bird populations. This damage cannot be undone. Thus the peregrine falcon was unintentionally exterminated as a breeding bird in the eastern United States. Damage to the liver and other suspected consequences are causing DDT to be prohibited today.

Thus we are confronted with quite new problems in the control of our waters. It seems to me that water analyses are almost of secondary importance. What should be tested is how effluent discharge into rivers and canals can be avoided. It will be necessary, in view of the difficulty in the determination and analysis of such substances in the water itself, to turn to an examination of the manufacturing process in the plant. It is of paramount importance, therefore, to control the production of industrial plants in the first place. If the control of these plants could be based on a detailed knowledge of the manufacturing processes and their waste products, it would be possible to obtain the maximum safety, and drinking water at minimum expense.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Imhoff, K. 1963. Taschenbuch der Stadtentwässerung.

Klotter, H.-E.; Hantge, E. 1966, Über die Auswertung biologischer Gewässeruntersuchungen und ihre Relation zum Biochemischen Sauerstoffverbrauch (BSB_5). *Wasserwirtschaft,* vol. 56, p. 21.

Reimann, K. 1966. Die Notwendigkeit der biologischen Abwasserklärung zur Reinhaltung der Gewässer in Erfüllung wassergütewirtschaftlicher und hygienischer Forderungen. *Österreichische Abwasserrundschau,* vol. 6.

Sladecek, V. 1965. The future of the saprobity system. *Hydrobiologia,* vol. 25, p. 518.

Tümpling, W. von 1967. Zusammenhänge zwischen Stoffhaushalt und Saprobiezustand bei Fließgewässern. *Fortschritte der Wasserchemie,* vol. 7.

DISCUSSION

Grindley: Is the period of five days for the BOD test sufficient for examining water from large rivers such as the Rhine? In the United Kingdom it is probable that any

discharge into a river reaches the estuary within five days and, if only the fresh water part of our rivers is considered, the five-day test is adequate.

When estuaries are considered the position is different. For example, sewage effluent from London cannot travel more than 50 k seaward in less than eighteen days, so we are interested in the ultimate oxygen demand rather than the BOD_5.

It is possible that a BOD_5 value for a sewage effluent is greater for a moderately good effluent than for a poor effluent because, in the case of the poor effluent, only a part of the carbonaceous matter is oxidized during the test, whereas for the better effluent ammoniacal compounds are also oxidized. The ultimate oxygen demand includes the oxidation of both compounds of carbon and nitrogen.

Hantge: Your question is justified since the organic substances are not actually decomposed within five days, particularly if you take into account that there are many paper mills situated along the length of the Rhine. However, the five-day period has generally been introduced in the Federal Republic of Germany for practical reasons to shorten the time for the analysis.

Grindley: Are you satisfied with the five-day period?

Hantge: No, but this is just an agreement.

Grindley: We would not get satisfactory results for the Thames estuary if we only had this short period of five days.

Measurement and significance of electrical conductivity in small mountain streams

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Abstract. Five small catchments in the Prealps of Switzerland were investigated over two years (1967-68) to establish relationships between discharge and some water quality characteristics. The catchments are located within the same general area of 20 km^2 and have a size of 0.5 to 1.5 km^2. Forests cover between 38 and 92 per cent, and swampy areas between 4 and 40 per cent. Their exposure varies between E and WNW. The electrical conductivity at 20° C, as measured with a plastic conductivity cell (cell constant 1.0 cm^{-1}) shows marked negative correlation with discharge rates as determined with the salt dilution method. The correlation coefficients (r) are $\geqslant$ 0.8 and the regression coefficients seem to be different for the five catchments. Assuming similar annual flow régime, the regression of electrical conductivity on discharge may be used to estimate average surface runoff conditions in these catchments.

MESURE ET SIGNIFICATION DE LA CONDUCTIVITE ELECTRIQUE DE L'EAU DES PETITS TORRENTS DE MONTAGNE

Résumé. Cinq petits bassins de réception des Préalpes suisses ont fait l'objet d'investigations intensives durant les années 1967-1968 afin d'établir les rapports existant entre quelques facteurs de qualité et les débits. Les bassins sont tous situés dans une même région d'environ 20 km^2 ; leur surface varie de 0,5 à 1,5 km^2. Les terrains boisés représentant 38 à 92 % des superficies, les zones marécageuses 4 à 40 %. L'exposition varie entre l'est et l'ouest-nord-ouest. On a mesuré la conductivité électrique spécifique à 20 °C à l'aide d'une cellule de conductivité en plastique (constante de cellule 1,0 cm^{-1}) ; la corrélation avec le débit est nettement négative. Le jaugeage du débit est effectué par la méthode chimique. Par nos calculs nous avons obtenu des coefficients de corrélation $r \geqslant 0{,}8$. Les coefficients de régression sont différents statistiquement pour les cinq torrents. En supposant des régimes annuels d'écoulement similaires, la conductivité électrique comparée aux débits peut alors servir à l'évaluation des valeurs moyennes de l'écoulement superficiel.

MEDIDA Y SIGNIFICACIÓN DE LA CONDUCTIVIDAD ELÉCTRICA EN PEQUEÑAS CORRIENTES MONTAÑOSAS

Resumen. En los años 1967-1968, cinco pequeñas cuencas de los Prealpes suizos fueron objeto de investigaciones intensivas a fin de establecer las relaciones existentes entre los caudales y algunos factores de la calidad del agua. Las cuencas estaban situadas todas ellas en una misma región de unos 20 km^2; su superficie varía de 0,5 a 1,5 km^2. Los terrenos con bosques constituyen del 38 al 92 %, los prados húmedos del 4 al 40 % de las superficies de cuenca, las vertientes dominantes varían entre el este y el oeste-noroeste. Se midió la conductividad eléctrica específica a 20°C con ayuda de una celula de conductividad de plástico (constante de la célula 1,0 cm^{-1}) y la correlación con el caudal es netamente negativa, el aforo del caudal se efectuó por el método químico y en nuestros cálculos hemos obtenido coeficientes de correlación $r \geqslant 0{,}8$. Los coeficientes de regresión demostraron estadísticamente las diferencias entre los cinco torrentes. Suponiendo regímenes anuales de escorrentía similares, la correlación de la conductividad electrica y el caudal se puede utilisar para estimar las condiciones superficiales medias de escorrentía en dichas cuencas.

ИЗМЕРЕНИЕ И ВЕЛИЧИНА ЭЛЕКТРОПРОВОДНОСТИ В МАЛЫХ ГОРНЫХ ПОТОКАХ

Резюме. Пять малых водосборов в предгорье Альп в Швейцарии исследуются в течение двух лет (1967/68) для установления отношения между расходом воды и некоторыми качественными характеристиками воды.

Водосборы сосредоточены в пределах одной общей площади в 20 км² и имеют площадь от 0,5 до 1,5 км².

Леса покрывают площадь между 38 и 92 %, заболоченные участки между 4 и 40 %. Их экспозиция варьирует между E и WNW.

Электропроводимость при 20 °C, измеренная пластической электропроводной камерой (постоянная камеры — 1.0 $см^{-1}$), показывает заметную отрицательную корреляцию с величинами расхода воды, определенными с помощью метода смешения при использовании соли.

Коэффициенты корреляции $r \geqslant 0{,}8$ и коэффициенты регрессии, вероятно, должны быть различными для пяти водосборов. Предполагая аналогичный режим стока, регрессия электропродности от расхода воды может быть использована для оценки средних условий поверхностного стока в этих водосборах.

INTRODUCTION

The electrical conductivity of stream water is an index of the amount of total dissolved solids. It is often used as a parameter of the chemical composition of stream and river waters (Durum, 1953: Hendricksen and Krieger, 1960: Kunkle, 1965). If concurrent determinations of certain ions are made, the electrical conductivity may be used to estimate the content of those ions which show a good correlation with electrical conductivity.

In regions with high precipitation and steep topography, runoff is relatively quick and there is frequent change between low flows and high flows, particularly in small catchments. Low flow water usually originates from deeply percolated waters and contains much more dissolved solids than high flow water. On the other hand, high flow water with a large percentage of surface runoff, consists almost entirely of pure rainwater. A model for this dilution process is given by Johnson *et al.*, 1969.

In the course of investigations on stream water chemistry in small mountain catchments in the northern Swiss Prealps (Keller, 1970) the effect of discharge on chemical composition of stream water is being studied. The present paper shows the measurement of electrical conductivity, its relationship to the discharge, and its possible use in describing the hydrological characteristics of a catchment.

THE STUDY AREA

Five small catchments in the northern Swiss Prealps (Alpthal, SZ) are the subject of this study (Fig. 1). Their characteristics are summarized in Table 1. Annual precipitation is about 2000 mm and the mean annual temperature is about 5 °C. Snow lies on the ground from November to April. Periods of snow melt may occur each month, and the maximum snow depth is usually observed in February with over 1.0 to 1.5 m of snow. Melt begins in late March after the snowpack has reached a snow water equivalent of usually more than 300 mm. In summer both frontal rains and thunderstorms are observed. Most of the summer storms have a marked effect on the

hydrograph of the streams. The parent rock is the partly impervious flysch formation and this may be assumed comparable in all 5 catchments.

Table 1. *Some characteristics of the five small catchments in the Alpthal (SZ)*

	Catchment				
	I	III	V	VII	VIII
Area, km^2	724	1.554	1.076	523	935
Altitude of gauging site, a.m.s.l.	1 250	1 060	1 110	1 120	1 110
Elevation range, m	300	500	450	450	400
Mean slope, %	33	35	34	54	30
Aspect	ENE	ESE	E	WNW	WNW
Forest cover, %	56	63	36	93	60
Swampy areas, %	37	25	27	4	40
Pasture, %	7	12	37	3	—

METHODS

For the catchments I, III, and VII two years (1967-68) of data are available, for VIII only one year. Water discharge, temperature, chemical composition and electrical conductivity were measured almost every week. Since rapid changes in the flow regime are observed frequently, discharge rates ranging from very low flow up to several hundred $1/sec \cdot km^2$ were sampled. At gauging site III, temperature and electrical conductivity have been recorded on a continuous basis since late 1967. For the discharge measurements the dilution method with sodium dichromate is used (Dodéro, 1952; Dumas, 1952). In the turbulent mountain streams this method yielded satisfactory results. Electrical conductivity is recorded in the field with a plastic conductivity cell (cell constant $1.0\ cm^{-1}$) attached to a transmitter and recording instrument. Temperature compensation to 20° C is built into the transmitter. In the laboratory, where determinations are made the day following sampling, electrical conductivity is measured with standard platinum electrodes. Both field and laboratory measurements reached a precision of ± 2 per cent. Laboratory measurements were on average, 13.5 μmhos/cm higher than those made in the field. This difference may be due to transportation of the sample, the instrument used, and the temperature compensation (stream water temperatures range between 0.5 and 12 °C, laboratory determinations are done at a room temperature of about 20 °C).

For the comparison of the five streams, the laboratory data are used as a base, while for the evaluation of the continuous record, some adjustments are necessary.

RESULTS

Electrical conductivity shows a considerable variation in time as well as between streams. The mean electrical conductivity at 20 °C varies between streams from 170 to 260 μmhos/cm ; the standard deviations (s_y) range from 38 to 50 μmhos/cm. The range of electrical conductivity within streams averages about 150 μmhos/cm.

Regressions using the model of

$$EC = a + b \cdot \ln Q \qquad (1)$$

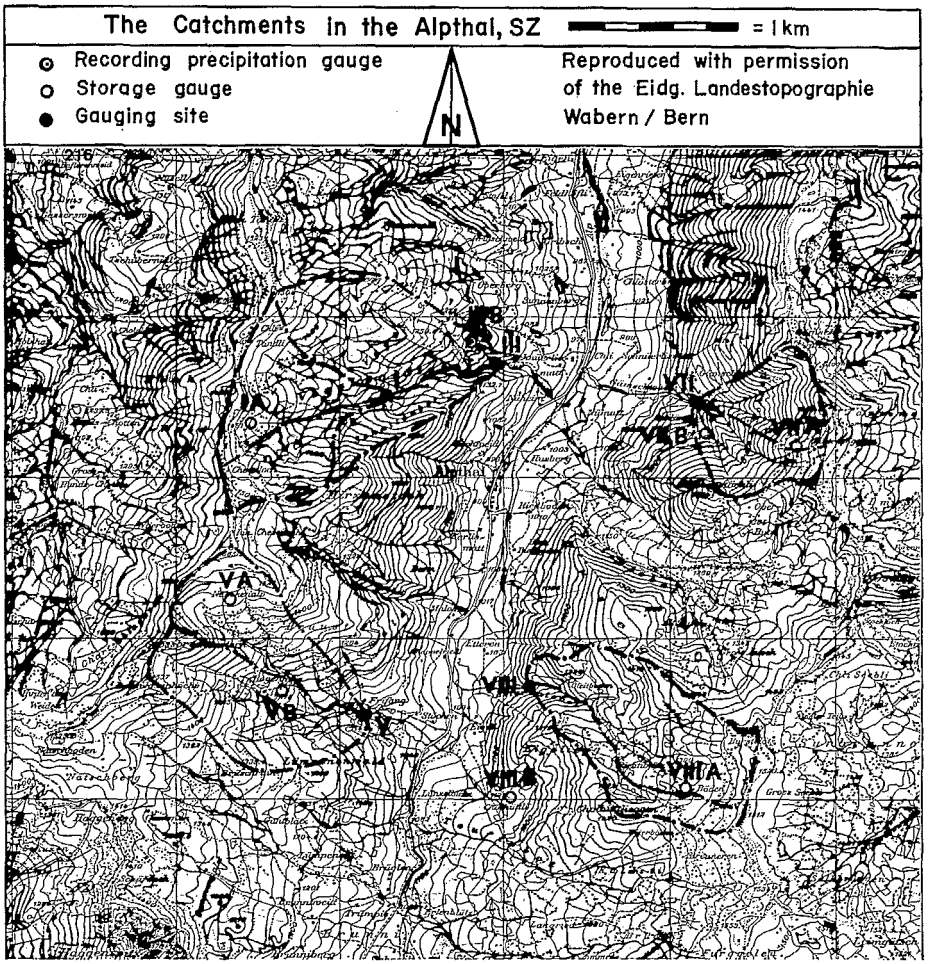


Figure 1 – *The catchments in the Alpthal, SZ*

are calculated, where EC is electrical conductivity at 20 °C in μmhos/cm, Q is discharge in l/sec . km², and a and b are constants. The results are given in Table 2. The standard deviations after regression ($s_{y.x}$) range from 18 to 26 μmhos/cm, or about half of that before regression. Regression constants a as well as regression coefficients b vary between streams (Fig. 2). Although the dilution effect is apparent in all streams, each stream has its own dilution response. Deep percolated water with a relatively high mineral content is discharged during baseflow and has the highest electrical conductivity. During and after rain or snowmelt, surface and subsurface waters are added to increase discharge rates. This addition of surface and shallowly percolated water of low mineral content brings about a dilution or decreasing concentration of dissolved solids and hence decreasing electrical conductivity with increasing discharge. The hydrological conditions of the soil and vegetation cover of the entire catchment area seem to have a significant effect on the shape of the dilution curve.

To estimate relative amounts of surface and ground-water runoff in total discharge, methods using stream water chemistry have been applied, e.g. by Kunkle (1965) and Pinder and Jones (1969). Calculations are based on the following relationship:

Table 2. *Regression analysis of electrical conductivity (umhos/cm) versus discharge Q (l/sec km²) in five small streams in the Alpthal, SZ*

Catchment/stream	Regression equation	r^2	n
I	$EC = 284 - 34.4 \ln Q$	0.83	59
III	$EC = 368 - 37.1 \ln Q$	0.87	164
V	$EC = 389 - 37.5 \ln Q$	0.73	88
VII	$EC = 356 - 28.0 \ln Q$	0.63	87
VIII	$EC = 288 - 30.1 \ln Q$	0.73	34

$$c_t - Q_t = c_g \cdot Q_g + c_0 \cdot Q_0 \tag{2}$$

where c_t is the concentration of dissolved solids in the total discharge, c_g is the concentration of dissolved solids in the ground-water discharge (baseflow), c_0 is the concentration of dissolved solids in the surface water, Q_t is the total discharge, Q_g is the ground-water discharge, and Q_0 is surface runoff. If

$$Q_t = Q_g + Q_0 \tag{3}$$

Q_0 as a percentage of the total flow is calculated

$$\frac{Q_0}{Q_t} \cdot 100 = \frac{c_t - c_g}{c_0 - c_g} \cdot 100. \tag{4}$$

If, for the concentration of dissolved solids, electrical conductivity is used, the following assumptions are made: c_0 is about twice the electrical conductivity of rainwater, or 30 µmhos/cm in all catchments; c_g is the electrical conductivity which corresponds to a flow of 6.6 l/sec. km² which is, in the average of the region, equalled or exceeded during 90 per cent of the year.

Using equation 4 we find for 6.6. l/sec km² no surface runoff, for a discharge of 35 l/sec km², Q_0 is 30 per cent in catchments I and III, 27 per cent in VIII, 22 per cent in V and 17 per cent in VII, and for 500 l/sec km², Q_0 is 79 per cent in I and III, 70 per cent in VIII, 57 per cent in V and 44 per cent in VII. These figures seem to indicate very clearly that surface runoff conditions, as determined with the electrical conductivity as a "tracer", are different between catchments. A check with conventional methods of separating surface runoff and baseflow was not possible, since continuous discharge measurements have only very recently been started in two of the fine catchments concerned.

The use of electrical conductivity to estimate discharge characteristics has also been attempted. A nearly continuous record of electrical conductivity at the gauging site of catchment III has been obtained and mean daily values computed. Since there was no check on continuous flow at the site, the gauging station of the 10 km² Grossbach basin (7 km NE) is used for comparison. In Figure 3 the two flow duration curves are shown for 1968. For Grossbach the flow frequency is taken from the Hydrographical Yearbook 1968 *(Eidg. Amt für Wasserwirtschaft, 1969)*. For catchment III, first the frequency distribution of mean daily electrical conductivity is

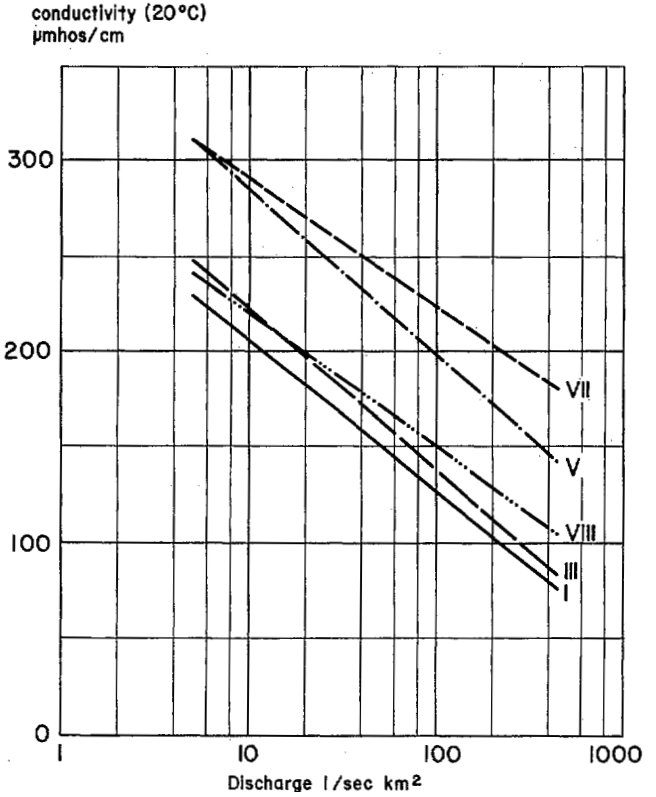


Figure 2 – *The relationship between electrical conductivity and discharge in the Alpthal catchments.*

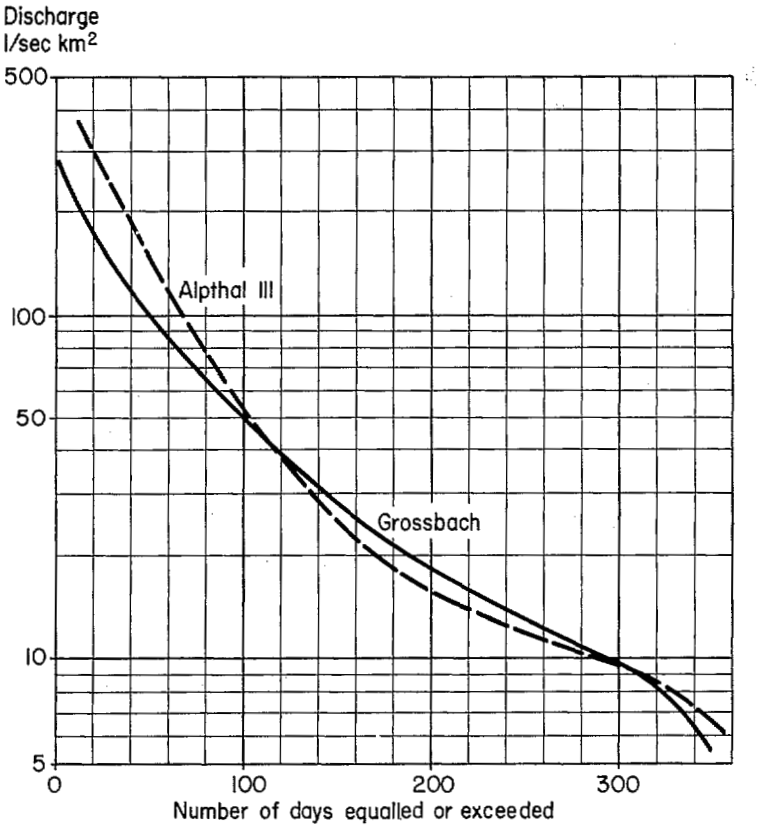


Figure 3 – *Flow duration curve 1968 for Grossbach as measured, and for Alpthal III as estimated from electrical conductivity data.*

calculated. Then using equation 1 (Table 2), the flow frequency is computed as shown in Figure 3. However, since field measurements do not agree with laboratory tests, the prediction equation using field data for electrical conductivity at III is adjusted to

$$EC = 271 - 29.7 \cdot \ln Q \tag{5}$$

with r^2 being 0.88, and the standard deviation s_{yx} being 13.8 μmhos/cm or 8.2 per cent of the mean. Except for high flows we find a close correspondence between the flow duration curve of Grossbach and the estimated curve of catchment III. Until we have a continuous flow measurement at the site, we cannot tell whether the difference is due to errors in the estimation of the flow, or to actual flow differences between Grossbach and III. Hence the estimation of daily and monthly flow rates is not attempted at this stage. Since there is no check on very high flows, and estimate of daily flows over the year would involve the use of the prediction equation beyond its range.

As shown in Table 2, 63 to 87 per cent of the variation of electrical conductivity can be explained in terms of flow rate differences (discharge). A further analysis with additional variables indicates that seasonal effects of water temperature in the stream, as well as the drying soil mantle in summer (evapotranspiration) and the cessation of moisture supply under a nonmelting snowpack in winter, may account for another 4 to 12 per cent of the variation. This leaves the unexplained variation of electrical conductivity as well as some other chemical components (calcium, carbonate) in the five catchments at 6 to 15 per cent.

CONCLUSIONS

From a study in the humid Prealps of Switzerland the electrical conductivity of the water is found to vary with the discharge rates. This relationship between electrical conductivity and discharge can best be described as a dilution effect which seems to be different between streams. Standard deviations after regression with discharge are about 8 per cent, which gives a basis for a fair estimate of discharge provided high flow rates can be included in the sampling.

The dilution curves may also be used to estimate relative amounts of surface runoff. The chemical analysis of the water samples by the Federal Institute for Water Supply, Sewage Purification, and Water Pollution Control in Zurich is gratefully acknowledged.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Dodéro, M. 1952. Sur l'emploi du dichromate de sodium dans la mesure du débit des cours d'eau par colorimétrie. *Comptes Rendus. Académie des Sciences,* Paris, p. 1462-4.

Dumas, H. 1952. La méthode chimique pour la mesure du débit des cours d'eau. La Houille Blanche, vol. 7, no. 5, p. 690-701.

Durum, W.H. 1953. Relationship of the mineral constituents in solution to stream flow, Saline River near Russell, Kansas. *Transactions American Geophysical Union,* vol. 34, no. 3, p. 435-42.

Eidgenössisches Amt für Wasserwirtschaft, 1969. *Hydrographisches Jahrbuch der Schweiz.* 1968. Eidg. Verkehrs–und Energiewirtschaftsdépartement, 275 p.

Hendrickson, G. E., Krieger, R. A. 1960. Relationship of chemical quality of water to stream discharge in Kentucky. *International Geological Congress, Copenhagen,* Report no. 1, p. 66-75.

Johnson, M.; Likens, G. E.; Bormann, F. H.; Fisher, D. W.; Pierce, R. S. 1969. A working model for the variation in stream water chemistry at the Hubbard Brook Experimental Forest, New Hampshire. *Water Resources Research,* vol. 5, no. 6, p. 1353-63.

Keller, H. M. 1970. Der Chemismus kleiner Bäche in teilweise bewaldeten Einzugsgebieten in der Flyschzone eines Voralpentales. *Mitt. Eidg. Anst. f. d. forstl. Versuchswesen,* vol. 46, no. 3.

Kunkle, G. R. 1965. Computation of groundwater discharge to streams during floods, or to individual reaches during base flow, by use of specific conductance. *US Geological Survey Prof. Paper,* 525-D, p. 207-10.

Pinder, G. F.; Jones, J. F. 1969. Determination of the Groundwater Component of peak discharge from the chemistry of total runoff. *Water Resources Research,* vol. 5, no. 2, p. 438-45.

DISCUSSION

Faber: 1. Why do you take a 2 per cent/°C temperature compensation?

2. The remark about the ionic composition being nearly constant may be correct for a very small watershed of uniform geological composition, but certainly is not true for a larger one.

3. What is the effect of suspended sediment load on electrical conductivity and do you compensate for it in some way ?

Keller: 1. For the temperature compensation we used a factor of 2.0 and this was in accordance with the tests in our catchments. This factor is applied in this transmitter by electric means.

Faber: I would like to know what kind of test it was because the electrical conductivity is influenced by the concentration you are working with, so the temperature compensation is not a constant.

Keller: 1. I may add that the temperature variation in these streams is not very high. They range in summer from about 12 to 13°C and in winter are approximately at 0.5°C.

2. Ionic composition of the water: a relative percentage of the various ions remains more or less constant during the various flows, even though the absolute concentration varies remarkably. In the streams which we investigated most of the ions are calcium and magnesium on the cation side and the carbonate on the anion side.

3. The effect of suspended sediments: we are aware of the fact that suspended sediments are influencing our readings. However, we are dealing here with a field calibration and automatically we include our readings which are influenced at high flows by suspended sediments. At the present stage I cannot give you any accurate figures on the effect of suspended sediments on the electrical conductivity.

Faber: But you do take them into account?

Keller: We do not. It is calibrated in the whole equation already in the black box.

Practical experience with devices to measure O_2 content, turbidity, solid matter content and electrical conductivity used for monitoring water quality in rivers

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Abstract. Emschergenossenschaft and Lippeverband operate automatic water quality stations. This paper describes practical experience with the O_2-electrodes, the measurement of turbidity and solid matter. The measurement of conductivity is important for control of salt load, control of industrial waste and observation of salts as tracers for flow measurement.

EXPERIENCES PRATIQUES EN VUE DE CONTROLER LA QUALITE DE L'EAU DES RIVIERES, AVEC UTILISATION DE DISPOSITIFS MESURANT L'OXIGENE DISSOUS, LA TURBIDITE, LES MATIERES SOLIDES DISSOUTES, LA CONDUCTIVITE ELECTRIQUE

Résumé. L'Emschergenossenschaft et le Lippeverband exploitent des stations de mesure équipées d'appareils automatiques. La note décrit l'expérience acquise dans la mesure d'O_2 par électrodes, la mesure de la turbidité et la mesure des transports solides. Il est important de mesurer la conductivité pour connaître la teneur en sels, pour contrôler des rejets d'eaux usées industrielles et pour utiliser les sels comme traceurs dans la mesure des débits.

EXPERIENCIAS PRÁCTICAS DE MEDIDA DE O_2, TURBIDEZ, MATERIAS SÓLIDAS Y CONDUCTIVIDAD ELECTRICA

Resumen. El Emschergenossenschaft y el Lippeverband explotan estaciones de medición equipadas con aparatos automáticos. Exponemos distintas experiencias prácticas con electrodos de O_2, así como las adquiridas en la medición de la turbidez y materias sólidas. La medición de la conductividad es importante con respecto al control del contenido de sales, control de aguas industriales y observación de sales.

О ПРАКТИЧЕСКОМ ОПЫТЕ ИЗМЕРЕНИЯ O_2, МУТНОСТИ, СОДЕРЖАНИЯ ТВЕРДОГО ВЕЩЕСТВА, ЭЛЕКТРОПРОВОДНОСТИ НА КОНТРОЛЬНЫХ УЧАСТКАХ ВОДОТОКОВ

Резюме. Ведомства рек Эмшер и Липпе располагают измерительными станциями, оснащенными автоматическим оборудованием. В докладе излагается практический опыт применения кислородных электродов, измерения мутности и твердых наносов.

Измерение электропроводности воды является в важным отношении контроля за содержанием соли в потоке, контроля за промышленными выпусками сточных вод и в ходе наблюдений за солью как индикатором при измерении расходов воды с помощью перемещающейся волны.

Since 1953, Emschergenossenschaft and Lippeverband have used continuously operating control stations to measure the temperature and oxygen content. These stations carry out continuous fish tests and continuously measure turbidity, solid

matter content, electrical conductivity and pH. Figure 1 shows the flow chart of such a station, and Figures 2 and 3 the inside of the Lippe-Wesel control station.

Experience in measuring oxygen content, turbidity, temperature and conductivity will now be described in detail.

OXYGEN CONTENT

Oxygen is important for chemical and biological processes in rivers. Yet a single oxygen measurement, even if taken continuously, does not give a direct index of the recuperative power of a river. But, of course, terms such as "recuperative power" or "load" of a watercourse should be considered in terms of a sufficient oxygen content in the river water. As the O_2-electrodes and their amplifiers are small, the dimensions of the equipment and hence the space required for continuous observation is also small. Figure 4 shows a section through an oxygen electrode covered by a membrane working on the polarographic principle.

Figure 5 shows the structure of an oxygen measuring station. An underwater pump draws about 100 l/min of water from the river and pumps it, free of bubbles, to an overflow tank in the measuring station. The rate of flow of water to the electrode is about 2 m/sec. The influence of the velocity of the water on the measurement depends upon the surface area of the cathode: the larger the cathode surface, the higher the required minimum flow velocity to the electrode. Practice has shown that this range is usually exceeded with velocities of about 1.5 to 2 m/sec, where contamination of the membrane gradually begins.

The working life of the membrane is from several months to one year, apart from accidental damage. In most cases the dirt cover could be removed carefully by a pad of cotton-wool. It is advisable to do this once a week when measuring river water and to check the calibration at the same time. With all the devices used, the calibration curves were linear. The service life of the electrodes was between 2 and 3 weeks, according to the type of electrode. Mechanical and temperature effects can cause an error of about 5 to 10 per cent, so that the calibration must be checked frequently. The pipes of the pump must also be freed occasionally from algae and bacteria which grow despite the high flow velocities. The water must be drawn from a well-mixed zone of the river and never from regions with little flow or strong plant growth, because, in the case of high solar radiation, much higher oxygen contents are found there than in the middle of the river or on the shady side.

Figure 6 shows the result of continuous oxygen measurements in a river with three measuring stations and the allocation of the measured values in accordance with the time of flow from a waste water discharge. The measurement was taken during the winter when biogene aeration was negligible.

TURBIDITY

When water contains settlable or floating solids, it appears turbid to the eye. In this case, the measurement of absolute values is of less importance than continuous observations of changes in the measured values over a defined range.

Experts differ in their choice of method between light absorption and light scatter, the first being more frequently used. The apparatus is designed according to the one beam, the two-beam, the differential or the compensation method. Cleanliness of the bulb is the important factor for all photographic measurements and

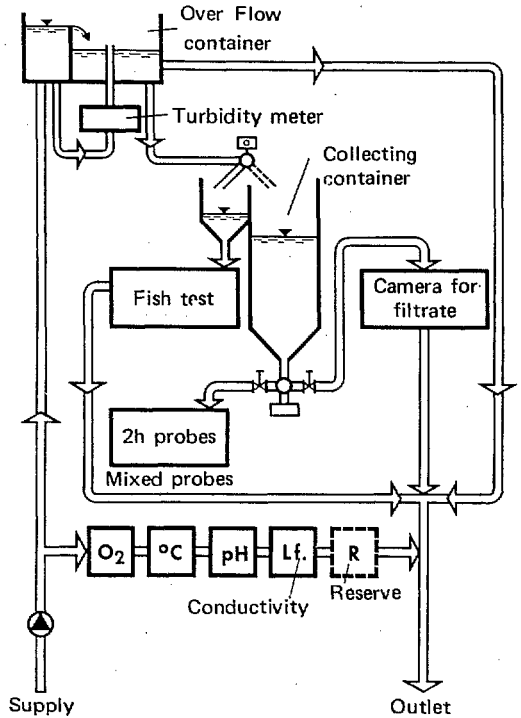


Figure 1 – *Flow chart of river monitoring station*

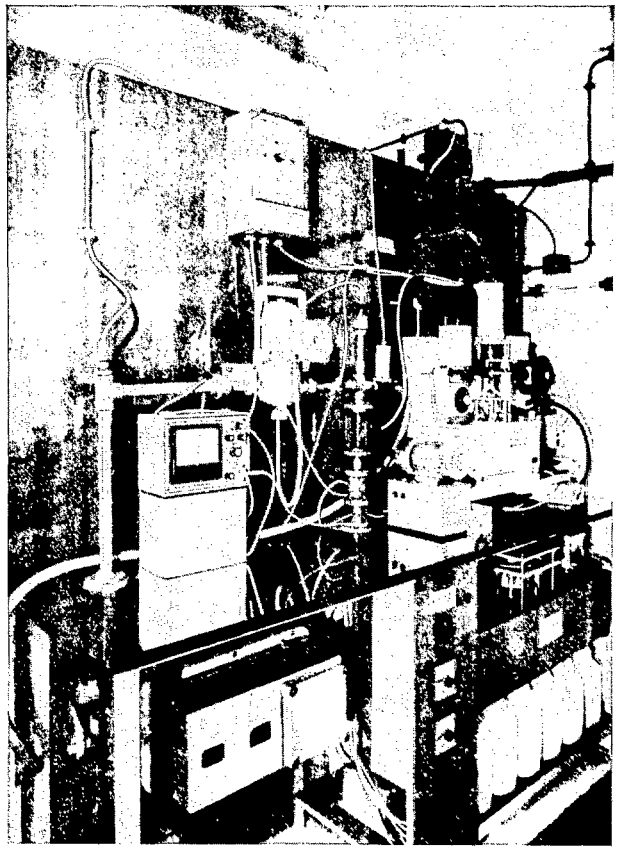


Figure 2 – *Lippe-Wesel monitoring station, sensor panel*

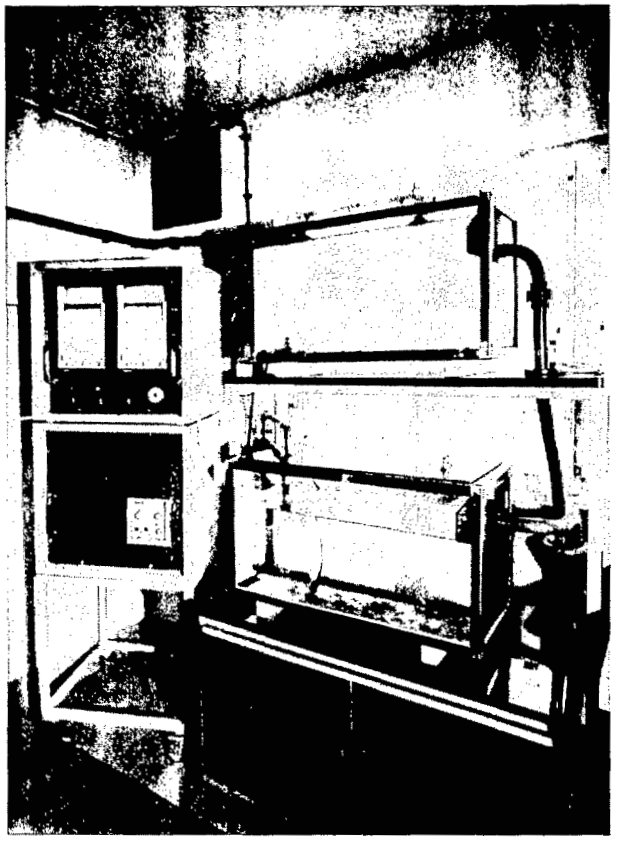


Figure 3 – *Lippe-Wesel monitoring station, biological fish test; recorder box*

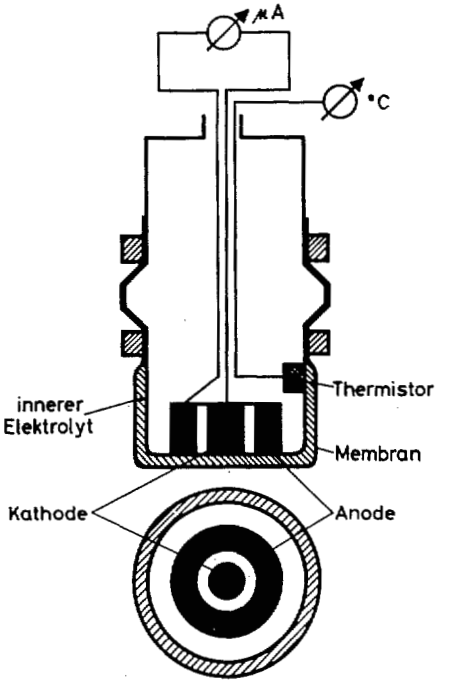


Figure 4 – *Membrane-covered oxygen electrode*

especially for continuous measurements. Cleaning can be done automatically by a double wiper. Automatic corrections of the zero point only partly solve the problem of pollution.

We shall now report experience with the Askania turbidimeter: a single beam photometer with a flow-through bulb (Fig. 7). Above the bulb chamber, is the control mechanism with a drive for the automatic cleaning of the cuvette. At intervals chosen by the operator, the wiper is moved into the bulb chamber and cleans the windows. Even after six years of operation, little trace of wiping could be found on the quartz windows. The working life of a bulb operated at less than normal voltage usually exceeds 10 000 hr. The instrument is calibrated in per cent of light permeability. Tap water is taken as the standard of calibration and reference and the permeability of light is set on 100 per cent. It is advisable to repeat the calibration every 1 to 2 weeks.

Assessment of the results, especially of the calibration, is difficult with turbidity measurements. Turbidity can be expressed in relation to the length of transparency (in cm), to the settlable solids content (in ml/l), to the quantity of solid matter (in mg/l) or also to specific standards. Light scatter, however, depends on certain properties of the solid matter such as size, colour, the amorphous or crystalline structure, distribution in the water, etc. With the heterogenous composition of sinking and floating solids in a river, these phenomena are so variable that a valid calibration in milligrammes of solid matter seems very doubtful. A reference, for instance, to transparency (in cm) appears to be quite reasonable for the interpretation of the measured values, and it gives a realistic picture of the visible purity of the water. This relationship must be found empirically for each measuring point. Measurements of turbidity are also relative measurements carried out to find and to record deviations from an empirical standard value and to give warning of these standard figures being exceeded.

As an example, Figure 8 shows two sections of recording tapes from the outlet monitor of a treatment plant. On the left-hand side, the outlet is normal. The peak at 10.40 hr corresponds to the light attenuation during the automatic cleaning of the bulb. On the right-hand side is seen a section from the beginning of a disturbance in the biological treatment plant, when activated sludge was pumped to the outlet.

SOLID MATTER

The solid matter content in water is differentiated, according to standardized methods, into sinking solids, filterable suspended matter and floating solids. We propose to begin with the method of volumetric measurement of settlable solids. There are two methods that can be applied: the photographic recording of the volume and the photo-electric measurement of the volume of solid matter settled after a pre-set period. With the photographic recording method (Fig. 9), 1 l of the water to be examined flows from a regulating vessel into a settling glass. After 2 hr, the glass is placed between a fluorescent screen and a special camera. A photograph is taken of the lower part of the settling glass, the film is wound on, the glass is cleaned with tap water and the equipment is then ready again for further measurements. A diagram is later obtained from the photographs in the form of dashes. The precision of measurement largely depends on the uniform shape of the surfaces of the settling glasses.

In the photo-electric method, the lower part of the settling glass, which is cylindrical, is employed. After the period of sedimentation, the height of the sludge

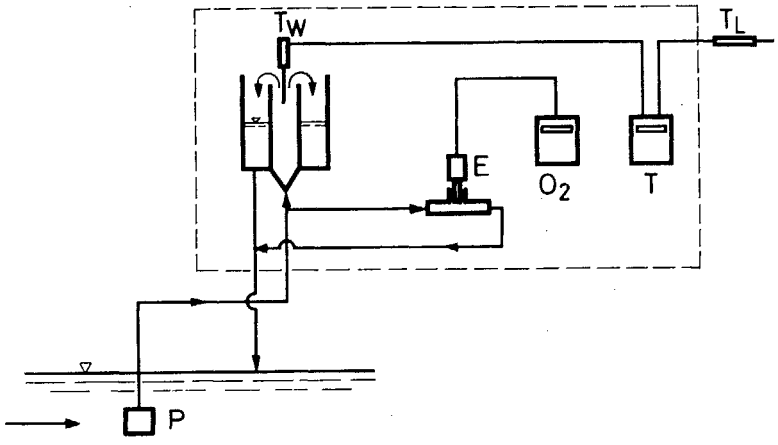


Figure 5 – *Flow chart of river oxygen monitoring system*
P = underwater pump
E = electrode
T_W, T_L = temperate measurement
O_2 = oxygen value recorder
T = temperature recorder

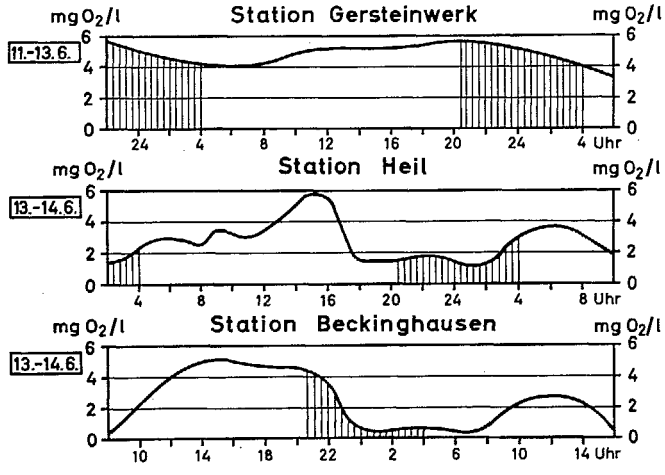


Figure 6 – *O_2-concentration of river water at three monitoring stations*

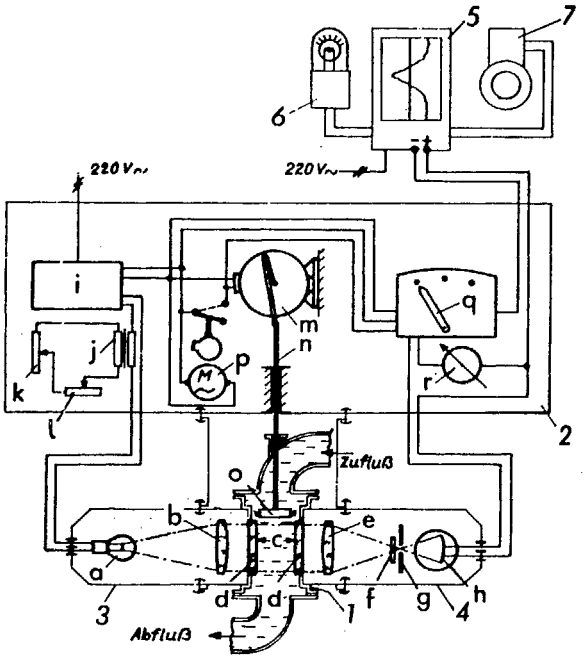


Figure 7 – *Turbidimeter with automatic cleaning of cuvette windows*

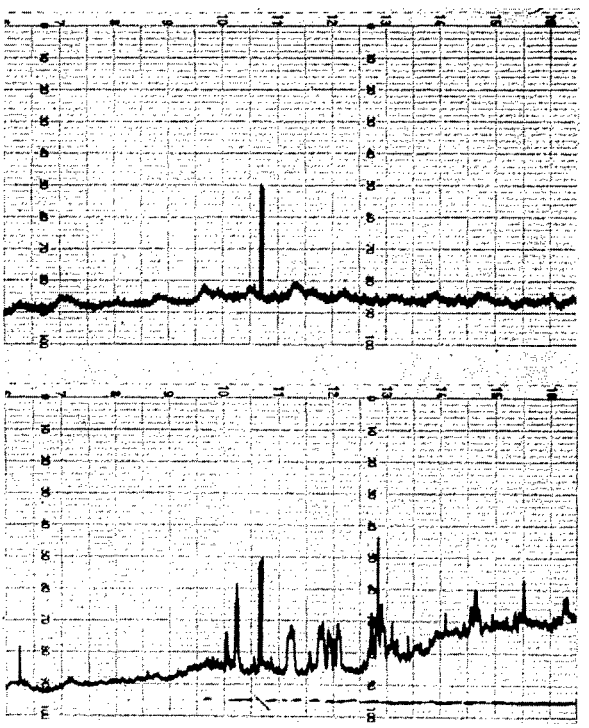


Figure 8 – *Visibility of biologically treated effluent (left : normal ; right : activated sludge in effluent)*

column in the settling glass is traced and transmitted to a recording device. The height of the sludge column is proportional to the volume of settled solids. The advantage of this method is that a direct measurement is obtained which can be transmitted (Fig. 10).

For a number of control purposes in rivers, the automatic production of stained filter papers is a valuable means of supplementing volumetric measuring of settlable solids and/or turbidity measurements. It is true that the latter permits conclusions as to what extent turbidities or sediments occur in water, but they give no indication about the source of the matter. The stained filter paper is well suited for this purpose. The possible sources of pollution can be assessed from the material on the filter paper as well as from the colour and structure of the filtrate, so allowing some semi-quantitative statements to be made about the pollution. Furthermore, the series of stained filter papers forms a simple record supplementing the turbidity or sludge volume measurement. Figure 11 shows this instrument. By means of a winding mechanism, a fixed amount of filter tape is pulled in at pre-set intervals, say every 15 min, across the suction device. During filtration the feed regulating device pushes the interceptor strongly against this suction device. After filtration, the suction mechanism lowers and releases the stained filter tape and the filter "picture" is then dried. After drying, fixing is carried out automatically and the time and date are noted on the series of filter "pictures".

ELECTRICAL CONDUCTIVITY

The continuous measurement of electrical conductivity enables rivers to be monitored for intermittent discharges of, for example, salt. It gives an indication of the presence of industrial wastes and it can be used for tracer measurements, and other water quality parameters.

The development of methods of measurement and of the structure of the electrode has continued from the Kohlrausch cell, which could be used only under certain conditions through the so-called electrode-less measuring cells which proved extremely good in practice, to the modern transmitters with four electrodes arranged in such a way that the pollution of the electrode surface is compensated over a certain range. These transmitters with four electrodes are designed either in the form of dipping devices or flow-through devices. Figure 12 shows a dipping sensor. This type of device must always be kept below the water surface and not nearer any side walls than 10 to 12 cm. By providing for mobile suspension of the sensor at angles under 45° in the direction of flow, the adhering of larger solid matter particles can be avoided. It is worth mentioning that no short-circuits or insulations are possible with the four-electrode systems, for instance when pieces of wire, plastic bags, oils or fats attach themselves to the transmitter. Maintenance is easy: the electrode surfaces can be cleaned by a brush and with the usual detergents. Some of the sensors are reversible in the measuring range between 1 μS/cm and 1 S/cm. For the most of them, temperature compensation is infinitely variable and is between 1.3 and 3 per cent/°C. Different types of sensor are commercially available, the temperature compensation of which is designed either for sodium chloride, potassium chloride or sodium hydroxide solution.

The electrical conductivity of the water depends on the electrolyte content. It is tempting to use, instead of S/cm, the dimension g electrolyte/l as a measure of

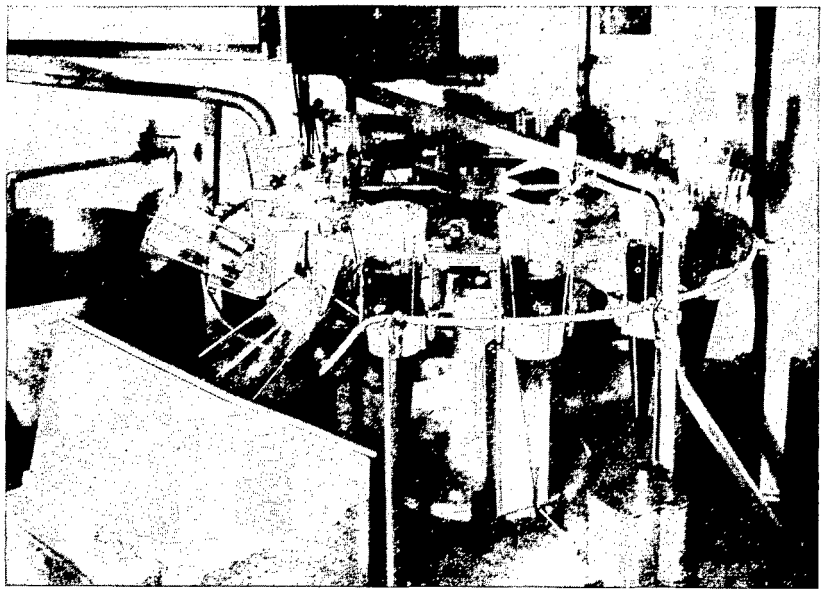


Figure 9 – *Photometering of suspended matter*

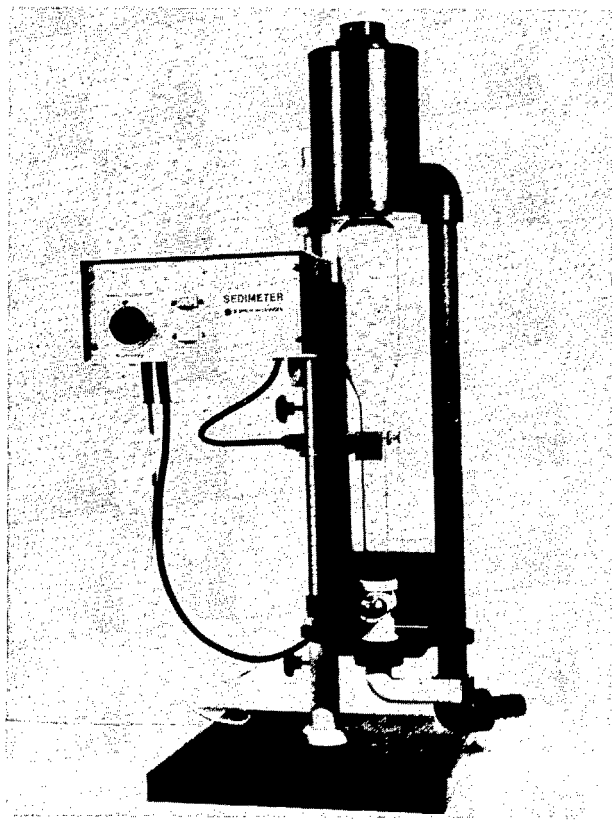


Figure 10 – *Photo-electric measurement of suspended matter*

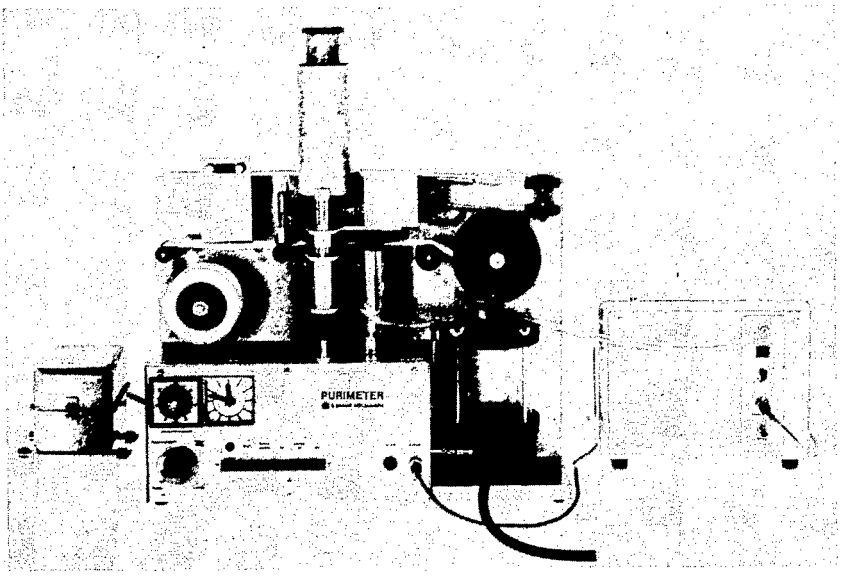


Figure 11 – *Filter "picture" instrument*

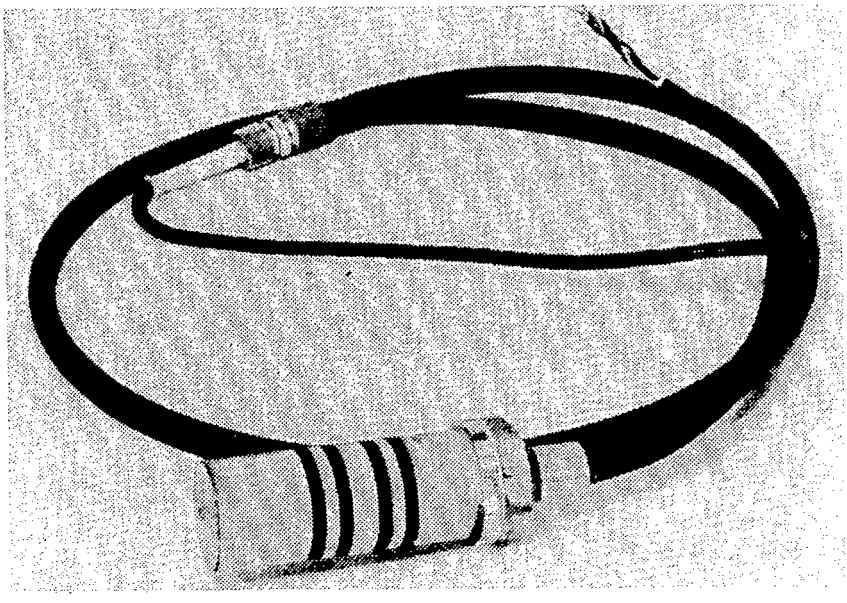


Figure 12 – *Four-ring electrode for conductivity measurements*

concentration as this is commonly understood. Calibration is carried out with solutions of known NaCl content, the conductivity of which is compared with the conductivity of the water subjected to investigation. It is often found with measurements of river water that there are good relationships between electrical conductivity and NaCl content and/or the residue on ignition. It was found in one case that the factor for the conversion from g electrolyte/l. to the actual NaCl content was approximately inversely proportional to the water level of the river in metres.

Continuous measurements of electrical conductivity in association with flow curves assume special importance in the estimation of other water quality parameters. Figure 13 shows the movement of a "flood wave" for a distance of more than 60 km, as indicated by variations in the electrical conductivity and/or NaCl content of this river characteristic for the time of the day.

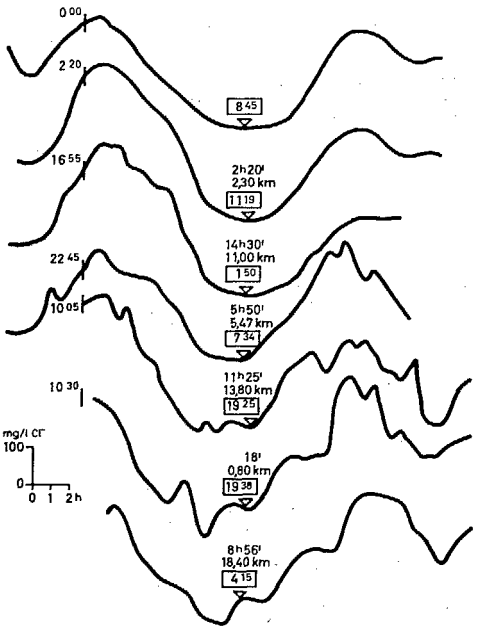


Figure 13 — *Following a flow wave by measuring the conductivity in river water*

CONCLUSIONS

When establishing monitoring stations on rivers, a decision must be made whether measurements are made in the water, i.e. whether the electrodes are put directly into the river, or whether the water is to be pumped to the electrodes installed in a suitable flow-through vessel. In our case we chose the latter course. In other words we pump the water by means of an underwater pump through a plastic pipeline to an overflow vessel in our measuring station.

The method of pumping the water is closely connected to the parameters to be determined. When measuring the oxygen content, pumping must be over a short distance with high flow velocities in order to avoid the entry of air, degassing or

oxygen consumption. When measuring turbidity or settlable solids, the pumping must be done in a way such as not to upset the water. As for measurements of the electrical conductivity or the pH, neither the length of the pumping pipeline nor the type of pumping have any influence on the results obtained. With very short high peaks of conductivity or pH, there is a possibility that these features will be affected in long pipelines. If the oxygen content is to be measured, in addition to other parameters, in a continuous measuring programme the pumping system must be especially adapted to the measurement of oxygen.

For all continuous measurements, the correctness of the figures measured depends on positioning of the measuring point. Thus, the location of measuring points should always be thoroughly considered and investigated, taking into account factors such as the distribution of inlets in the cross-section, backwater effects from the next tributary, the conditions of flow and protection against high water. Of great importance is the ease of access to the measuring point. In the case of long period and/or continuous measurements, the quality of maintenance of the instruments depends largely on their accessibility.

The construction of the continuously operated analysis devices must fulfil certain requirements. The instruments must be robust in design. Cleaning must be easy, and they must be able to withstand climatic and corrosive influences. The measuring principle should enable operation for weeks and even months without much attendance. We are aware of the fact that it is difficult to fulfil these demands in practice. We observed time and again that measuring devices, which work without defects for years in boiler feed waters, drinking waters, condensers, etc., fail in effluents or in river water. These defects are mainly due to blockages in the water inlets and the valves or to soiling of the electrodes and bulbs.

The colorimetric front-end sensors in automatic surveillance of water quality

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Abstract. In the instrumentation systems used for continuous, unattended, surveillance of water quality, the following components are recognized: front-end sensors, signal conditioners, data transmitters, data storage/retrieval components and data conditioners. The characteristics of the front-end sensors may also require sample conditioners. The front-end sensors and sample condtioners are often the weakest components of the overall instrumentation system. The lack of compatible front-end sensors is more pronounced when data are utilized in system automation or process control. In the surveillance of water quality for measurement of many parameters at the present time, colorimetric modules are the reliable and recommended type of sensors. For the optimum performance of the continuous, unattended colorimetric module in many applications an equally automatic filtration system is a necessity. This paper describes the design aspects of the continuous, unattended, colorimetric modules and sample conditioners and suggests techniques for obtaining over-all system compatibility.

CAPTEURS FRONTAUX COLORIMETRIQUES UTILISES DANS LA SURVEILLANCE AUTOMATIQUE DE LA QUALITE DE L'EAU

Résumé. Dans les systèmes utilisés pour la surveillance de la qualité de l'eau, on distingue les éléments suivants : capteurs frontaux, convertisseurs de signaux, transmetteurs de données, archivage, traitement et transmission des données. Les caractéristiques des capteurs frontaux peuvent aussi nécessiter des convertisseurs de signaux. Les capteurs frontaux et les convertisseurs de signaux sont souvent les parties faibles de tout le système. Ces faiblesses sont encore plus prononcées quand les données sont utilisées dans des systèmes automatiques de contrôle et de traitement. Cette étude décrit : (a) une variété de capteurs frontaux actuellement utilisés ou en cours de développement en mettant l'accent sur les modules colorimétriques et les convertisseurs de signaux ; (b) modification préconisées et proposition de nouvelles techniques pour obtenir l'objectif désiré qui est la totale comptabilité du système.

EL SENSOR COLORIMÉTRICO FRONTAL PARA OBSERVACIÓN AUTOMÁTICA DE CALIDAD DEL AGUA

Resumen. En los sistemas utilizados para la vigilancia continua de la calidad del agua, se identifican los siguientes componentes : extremo frontal de los detectores, acondicionadores de señal, transmisores de datos, componentes de almacena miento/retirada de datos y analizadores de datos. Las características del extremo frontal de los detectores pueden también exigir acondicionadores de muestra. El extremo frontal de los detectores y acondicionadores de muestra son con frecuencia las partes débiles de todo el sistema. Este problema es todavía más acusado cuando los datos se utilizan en un sistema automático o en un proceso de control. El presente estudio describe : (2) una variedad de detectores de extremo frontal actualmente utilizados o en desarrollo especialmente los módulos colorimétricos y acondicionadores de muestras, (b) cambios propuestos y utilización de nuevas técnicas para obtener el objetivo descado, que es la total compatibilidad del sistema.

ДАТЧИКИ ДЛЯ НЕПРЕРЫВНОЙ РЕГИСТРАЦИИ КАЧЕСТВА ВОДЫ

Резюме. В аппаратуре для непрерывной регистрации качества воды имеются следующие элементы : датчики, преобразователи сигналов, передатчики данных, накопители данных и преобразователи данных. При определенных технических характеристиках датчиков может возникнуть необходимость в преобразователях взятых проб.

Датчики и преобразователи проб часто являются наиболее уязвимыми звеньями системы. Эта уязвимость проявляется особенно сильно, если данные используются в автоматических или контролируемых системах обработки. В докладе описываются: (1) различные типы датчиков, применяемые в настоящее время или разрабатываемые, причем особое внимание уделяется колориметрическому методу и преобразователям проб, (2) предложения в отношении новой методики для получения желаемой точности наблюдений и надежности системы.

AUTOMATIC COLORIMETRIC ANALYSERS

In water quality analysis, the accepted reference in the U.S.A. is *Standard Methods for the Examination of Water and Wastewater* (1965). These procedures are written for discrete laboratory analysis. To optimally apply the standard methods to instrumentation for continuous, unattended analysis of water and waste water, new design criteria should be considered. The generalized objectives of the continuous front-end sensors are: accuracy, repeatability, long period of unattended operation, capability of measuring a wide band of variation, fast response time, economy, ease of maintenance in the field and safe operation. The objective function has n variables that are not necessarily independent. For any given set of conditions, this function can be optimized to give the best combination of the variables.

The over-all functional behaviour of a continuous colorimetric analyser is basically the relevant knowledge governed by the following three disciplines: chemistry, hydraulics, and electro-optics.

Chemistry of the continuous colorimetric analysers

Objective: Selection of a dependable and well-accepted chemical procedure that would yield a unique colour intensity as a function of the magnitude of the desired parameter with a minimum of interference from other constituents.

The chemical variables given by *Standard Methods* include: number of reagents, their chemical characteristics, ratios of the reagent volume to the sample volume, the optimum environment for chemical interaction, reaction time, other expected interferences, optimum wave-length, its bandwidth, recommended cell length or the light path for different ranges of the concentrations.

After obtaining the pertinent chemical information, a series of tests should be run to evaluate the effects that the variation of the above chemical variables, such as sample to reagent flow rates, time of reaction and temperature, would have on the colour reaction.These tests are a major component on evaluating an instrument's complexity and its technical capability. In most applications, the well-known Bouguer-Beer law gives the necessary relationship for evaluating the concentration of the constituents as a function of the intensity of the specific colour formation.

Hydraulics of the continuous colorimetric analysers

Objectives: To provide a constant and repeatable flow rate for long periods of unattended operation, to generate proper mixing among reagents and sample, to minimize delay time, response time and longitudinal mixing and to provide bubble-free fluid flow in the flow cells.

At the present time in many applications, hydraulic limitations determine the length of time that an instrument can be operated unattended. The common means for delivery of a fluid with a constant flow rate are: (a) constant head devices, (b) capillary tubes, and (c) pumps.

Constant head devices. These devices maintain a constant head that is equal to the local atmospheric head plus a head corresponding to a constant height of the fluid. Occasionally, pressurized capsules are used to replace atmospheric pressure on the influent side of the system. If the sampling stream is not free of particulate matter, the flow rate will vary significantly in time. Biological growth, sedimentation and chemical precipitation are also restricting factors in the flow rate consistency. Generally, constant head devices are used where the physical properties of the fluid are controllable such as in the reagent flow. The common range of flow rate is in the order of a few cm^3/hr.

Capillary tubes. For a given fluid and head, the flow in a capillary tube is primarily a function of the tube length and its inside diameter. This means of fluid delivery, as in the previous case, is usually used for small flow rates and where the physical and the chemical properties of the fluid can be controlled, i.e., reagent flow. The common range of flow rate is in the order of 5 cm^3/hr.

Pumps. The multi-channel peristaltic pumps are commonly used for fluid flow in colorimetric analysers. For a given fluid at a constant temperature, the flow rate in each channel of a peristaltic pump is primarily a function of the inside and outside diameters of the tube as well as the rotation per minute and other properties of the motor. Flow rate is also a function of the elastic properties of the material of the tube. A multi-channel peristaltic pump, if designed and operated properly, can keep the ratios of the reagents to sample flow rate relatively constant for extended periods of time. Any individual channel during this period of operation might have a significant flow rate variation. The period of unattended operation is one to two weeks.

To minimize delay time, response time, and longitudinal mixing, attention should be given to the flow rate, the length of the tubing, over-all volume of the system and the inside diameter of the flow passages. The minimum volume necessary for the system is given by the chemical reaction rate, the volume of tubing and other flow passages, and the optimized flow rate.

Dual beam colorimetric analysers are more desirable. In a dual beam colorimeter the unreacted sample passes through a reference flow cell while the reacted sample passes through the sample flow cell. The reference flow cell automatically corrects the output of the instrument for disturbances caused by variations in the light source and the initial colour of the sample stream. The correction due to variation in colour of sampling stream is more efficiently performed if the photodetectors are sensing the same sample of water at a given time.

In the design of the flow cells, special attention should be placed on the bubble release capability. The gas bubbles in the flow cells of the colorimetric modules interfere with the passage of the light. After the initial starting period, gas bubbles can exist in the flow cells only due to degassing of the sampling stream while passing through the cells or malfunctioning of the bubble release mechanism. To eliminate

degassing of the sample stream, the sample may be degassed by heating prior to the entrance to the flow cells. At the same time, the heating mechanism may be also utilized to minimize the chemical reaction time. To increase gas carrying capacity of the fluid, flow cells may also be pressurized.

Electro-optics of the continuous colorimetric analysers

Objectives: The selection of a light source that emits optimally for long periods of time around the wave-length of interest, proper optical filters, and photodetectors that have unique (preferably logarithmic or linear) response to the variation of the light intensity around the wave-length of interest.

A light source should have a consistent spectral curve and should be economically feasible. For applications in the visible or infra-red ranges, filament type lamps are commonly utilized. The emission rate of the lamps is optimized, for example, by iodine cycle process, developed by General Electric. The iodine cycle process causes most of the evaporated tungsten to deposit in areas of the filament where minimum interference will result. When operating the lamps below their rated voltage, it is not unlikely to obtain a useful life of more than a year.

For isolating the wave-length of interest in continuous colorimetric analysers, interference type filters are commonly used. The interference filters are available in many wave-lengths, bandwidths and different types of background rejection characteristics.

The photodetectors should have a unique electrical response, preferably linear, to the intensity of the incident filtered light. At Automated Environmental Systems, Inc., phototransistors are being utilized to detect the light level.

FILTRATION SYSTEM

In many applications to implement an unattended continuous colorimetric analyser, it is desirable to have at least an equally automated filtration system. The filtration system minimizes biological growth, clogging and sedimentation in the flow passages and optimizes flow cell performance; i.e., it extends the length of unattended operation. In the design of an automatic filtration system, dead volumes should be minimized in order to minimize delay and response times, and to optimize the sensitivity. As an example of such a system, AES 0.35 filtration system is described in the following paragraph.

AES 0.35 filtration system is composed of two stages (Fig. 1). The first stage is a zero dead volume self-cleaning filter that eliminates all particles larger than 5 microns nominal from the sampling stream. The second stage is composed of parallel, cartridge-type, disposable filters with negligible dead volumes. The number of disposable cartridge filters is a function of the stream condition, the required reliability and the length of unattended operation desired. At any given time only one cartridge filter is in the line. A pressure-sensitive valve diverts flow from one cartridge filter to the next fresh one when head loss across the cartridge filter exceeds a prescribed value. The second stage filter eliminates all particles larger than 0.35 or 3 microns absolute from the sampling stream according to the specific demand. The over-all volume of the system is about 50 cm^3. AES filtration system is capable of maintaining a filtered sample flow rate of up to 750 cc/hr for long periods of unattended operation.

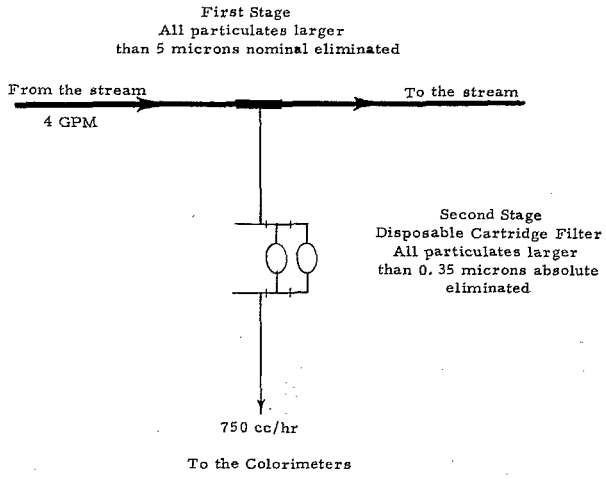


Figure 1 – *AES 0.35 filtration system*

BIBLIOGRAPHY / BIBLIOGRAPHIE

Standard Methods for the Examination of Water and Wastewater (12th edition). 1965. New York, American Public Health Association.

DISCUSSION

Smoot: How much is the severe filtering to three microns affecting the samples?

Zand: This depends on the specific parameters which are measured. In some series of tests for measuring tetravalent chromium we studied the effect of eliminating particles from the sample stream and we more or less arrived at an effect which could be predicted.

Wilson: 1. Does measurement of an unreacted sample against a chemically processed sample lead to errors due to chemical additions?

2. Would you please give more details of self cleaning filtration systems ?

Zand: When I was working in the company which produced this colorimetric analyser, we had a unit for phosphate. We were detecting about 820 Å for that wavelength.

VIII Radiohydrometry
Radio-hydrométrie

Deuterium and oxygen-18 measurements on surface waters of the Bavarian Prealps

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Abstract. The hydrogen isotope ratio and the oxygen isotope ratio in brooks, rivers and lakes in the Bavarian Prealps are reported. The results are discussed with regard to the origin of the water and its intermixture, as well as to special isotope fractionations during flow time. Under certain conditions, isotope measurements can furnish quantitative hydrometric results, e.g. for the evaluation of components of flow systems.

MESURES DU DETERIUM ET DE L'OXYGENE-18 DANS LES EAUX DE SURFACE DES PREALPES BAVAROISES

Résumé. Cette communication donne une description des études effectuées sur les proportions d'isotopes d'hydrogène D/H et d'xygène 0-18/0-16 contenus dans les ruisseaux, fleuves et lacs des Préalpes bavaroises. Les résultats font l'objet d'un examen critique qui tient compte de l'origine de l'eau, des mélanges des eaux, ainsi que des fractionnements spéciaux d'isotopes pendant l'écoulement. Les auteurs montrent qu'éventuellement ces mesures d'isotopes peuvent fournir des données hydrométriques quantitatives (par exemple : la répartition des débits aux confluents).

MEDICIONES CON DEUTERIO Y OXÍGENO-18 EN AGUAS SUPERFICIALES DEL PAÍS ALPINO BÁVARO

Resumen. Se presenta una comunicación sobre la relación isotópica del hidrógeno y del oxígeno en torrentes, ríos y lagos del país Alpino Bávaro. Se discute el resultado con respecto al origen del agua y su intermezcla así como a fraccionamientos isotópicos especiales durante el *flow-time*. Se señala que bajo determinadas condiciones las mediciones isotópicas pueden proporcionar resultados cuantitativos hidrométricos (p. ej., para la evaluación de componentes de sistemas de flujo).

ИЗМЕРЕНИЯ ДЕЙТЕРИЯ И КИСЛОРОДА-18 В ПОВЕРХНОСТНЫХ ВОДАХ ПРИБРЕЖНОЙ ПОЛОСЫ БАВАРСКИХ АЛЬП

Резюме. Доклад посвящен вопросу соотношения изотопов водорода и кислорода в ручьях, реках и озерах прибрежной полосы Баварских Альп. Результат обсуждается относительно источника воды и ее примеси, а также специальных изотопных частиц в течение периода стока. Будет показано, что при определенных условиях изотопные измерения могут дать количественные гидрометрические результаты, например, для оценки компонентов водных систем.

INTRODUCTION

In addition to the light isotopes of hydrogen (^{1}H) and oxygen (^{16}O), deuterium ($D = {}^{2}H$) and ^{18}O are present in the water molecule. According to the data gathered so far, the D and ^{18}O content of natural waters, related to the number of hydrogen

and oxygen atoms in the water. varies between 170 and 100 and 2,010 and 1,920 ppm, respectively. These fluctuations result mainly from evaporation and condensation processes, but also from diffusion and exchange processes and changes of state in the course of the hydrological cycle. From the D and ^{18}O content, therefore, certain conclusions may be drawn about the origin of a water sample. Thus, for example, the D and ^{18}O content of precipitation depends on the geographical latitude, on the orographic altitude of the precipitation area and on the season. This is due, in essence, to the lower vapour pressure of $HD^{16}O$ and $H_2{}^{18}O$ compared with $H_2{}^{16}O$.

The D and ^{18}O content of water samples is usually given in terms of the deviation δ of the isotope ratio of the sample, R_{sample}, from the isotope ratio of standard water, $R_{standard}$:

$$\delta = \frac{R_{sample} - R_{standard}}{R_{standard}} \cdot 1\,000\ (‰). \tag{1}$$

As the customary international standard, an ocean water sample according to Craig (1961), the so-called SMOW (standard mean ocean water), is used. Between the δ-value of D, δD, and the δ-value of ^{18}O, $\delta^{18}O$, there exists a linear relationship

$$\delta D = m \cdot \delta^{18}O + t. \tag{2}$$

The slope m generally has a value of about 8. Lower values indicate that ^{18}O underwent a greater enrichment than normal compared with D. This occurs, for instance, during rapid (kinetic) evaporation of water. The constant t has a value in the central European area of about + 10. Also in this case, deviations point to several physical conditions in the water cycle of the sample examined. For further details, reference must be made to the literature (for example, Dansgaard, 1964 ; Friedman et al., 1964).

Taking into account the relationships mentioned, D and ^{18}O measurements on surface waters can furnish indications of the location of the catchment areas of these waters. Thus it has been possible to detect, for rivers of different countries, a clear dependence of the δD value on geographical latitude and orographic altitude (Friedman *et al.*, 1964 ; Eichler, 1964). Correlations of the isotope contents of precipitation, ground and surface waters can be used in hydrographic investigations and for a determination of the pattern of variation with respect to groundwater (Guidebook on nuclear techniques in hydrology, 1968). There is a possibility of calculating the respective discharges of two rivers at a confluence from their differing isotope content (see Section 4). Isotope measurements on lakes can contribute towards knowledge of the balance of surface and subsurface inflow and outflow (Guidebook, 1968 ; Zimmerman, 1969). The $\delta^{18}O/\delta D$ relationship lends itself to a determination of the evaporation from a lake surface (compare, for example, Zimmerman, 1969).

The present paper discusses the isotope content of the surface waters in the southern catchment area of the Isar river (Upper Bavaria). The results demonstrate that measurements of D and ^{18}O contents, even in a relatively restricted area, can furnish quite interesting hydrological findings.

MEASURING METHOD

The only way to determine, with the required accuracy, the fluctuations of the D and ^{18}O isotope contents which are of hydrological significance is by employing a mass spectrometer. Since water vapour causes high memory effects in the mass spectrometer and thus cannot be used as measuring gas, the water sample for the D measurement is decomposed quantitatively to hydrogen as the measuring gas. The customary set-up utilized for this purpose is described in Friedman (1953) and Horibe and Ogura (1968). About 0.1 ml is required as the sampling quantity. For the ^{18}O measurements, an oxygen isotope exchange is effected between the sampling water and CO_2, this CO_2 then being introduced into the mass spectrometer as the measuring gas. A description of the apparatus used can be found in Dansgaard (1961). The sampling amount required is about 10 ml. The mass spectrometers employed are the models M 86 and GD 150 of the firm Varian-MAT, Bremen ; these are specially equipped for isotope ratio measurements on gases. With regard to details of the equipment and of the measurements, including the necessary corrections, reference must be made to the descriptive brochures of the firm and to Brunee and Voshage (1964) and Craig (1957) for example. The absolute accuracy allowed by sample preparation and measurement is ± 1 per thousand for δD and ± 0.2 per thousand for $\delta^{18}O$. The δD and $\delta^{18}O$ values given in the following section have been related in all cases to SMOW.

MEASUREMENTS

Figure 1 shows the southern catchment area of the Isar river (Upper Bavaria) from which water samples of 20 ml each were taken at 30 to 40 measuring points. Sampling was carried out in four measuring series on 8 and 9 November 1968, 27 February 1969, 23 June 1969 and 21 November 1969. In addition, samples were collected in certain sub-areas (at Sylvenstein-Stausee, for example) on other days.

All water samples were analysed for their D content. Measurements of the ^{18}O content were performed particularly in those cases where possible deviations were to be expected from the normal values of equation 2 ; for example, on water samples from lake inflows and outflows. As an example, the δD values of the series of measurements made on 8 and 9 November 1968 have been given in Figure 1 for the respective sampling points.

RESULTS AND DISCUSSION

The survey of δD values presented in Figure 1 shows a steady increase of the D content along the watercourses. The waters with the lowest δD values were found in the source areas of the high mountains (mean value, about – 93 per thousand), since there most of the water is freshly fallen rain or snow ("isotope altitude effect" ; compare, for instance, Eichler, 1964; Moser and Stichler, 1970). On the other hand, measurements on snow samples in the vicinity of the Isar spring in the Karwendel Range furnished an average value of $\delta D = 105$ per thousand, which is lower than the δD value for the Isar in this area; however, a significant enrichment in D occurs during the ageing and thawing of snow (see Moser and Stichler, 1970).

Figure 2 shows the δD values for some tributaries of the Isar as a function of catchment area elevation, the altitude range from the source to the mouth being

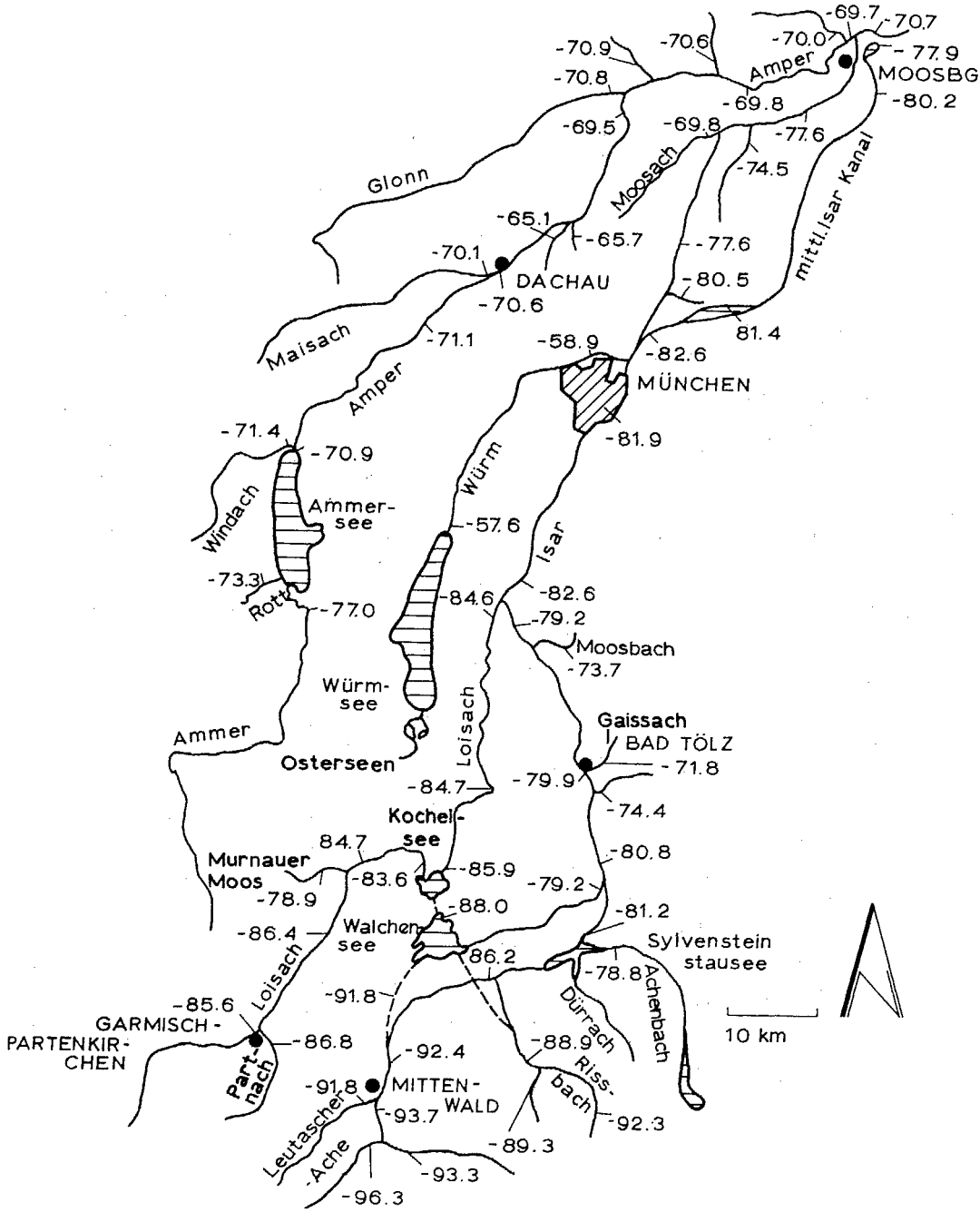


Figure 1 – *Southern catchment area of Isar River (Upper Bavaria), . . . = water tunnels (penstock and diversion tunnels). The figures represent δD values of the water samples taken at the indicated points of measurement on 8 and 9 November 1968*

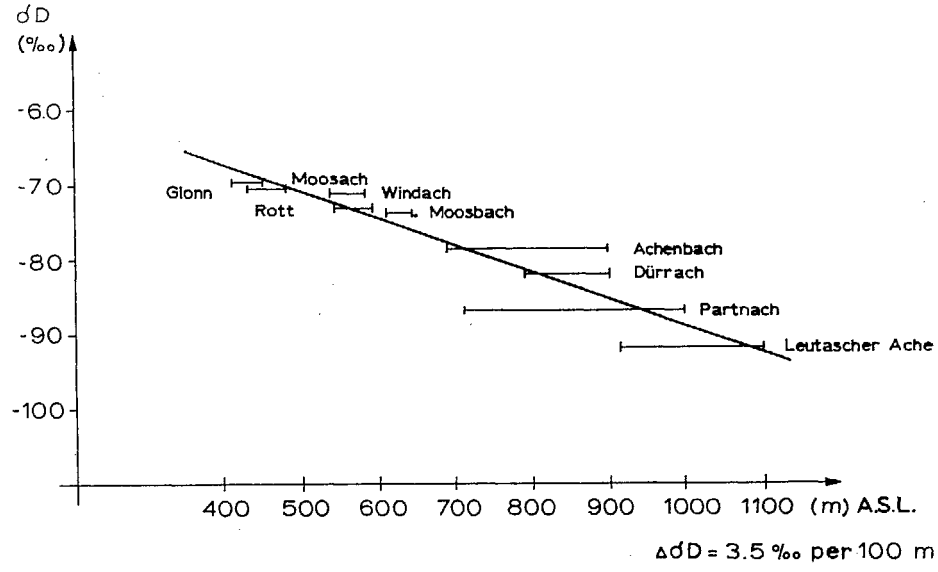


Figure 2 – *δ D values for some tributaries of the Isar draining watersheds at different elevations. The length of the horizontal lines corresponds to the altitude range of the watershed from the source to the mouth of the respective tributary. Sampling on 8 and 9 November 1968*

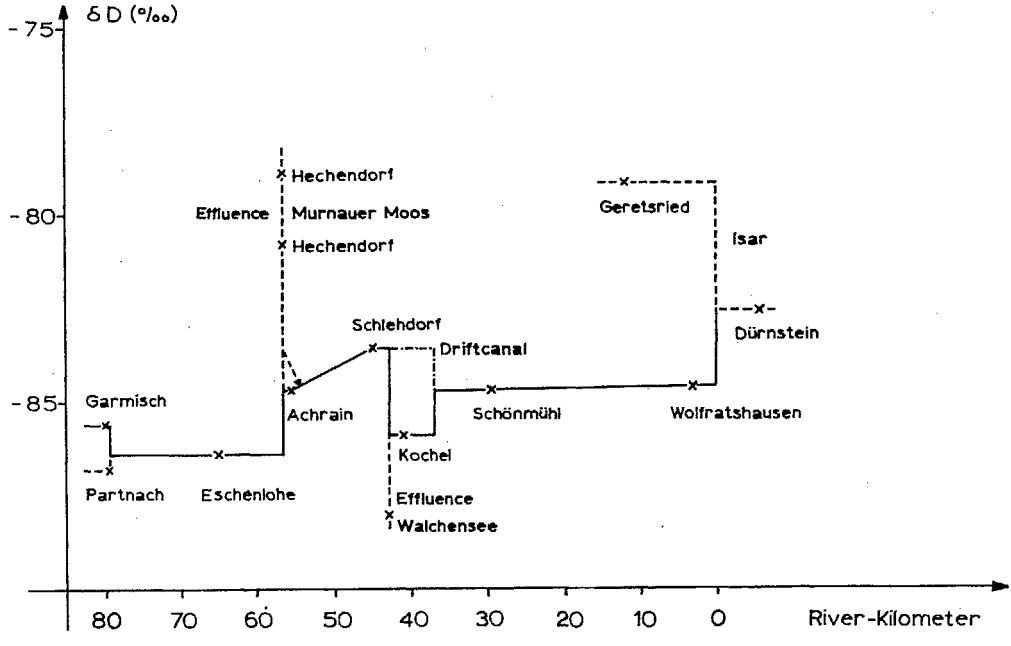


Figure 3 – *Schematic representation of the variation of the D content of the Loisach along its course. Sampling on 8 November 1968*

Table 1. *Percentage of Loisach water and Isar water in the Isar after the Loisach and the Loisach-Isra-Kanal have joined the Isar. The figures were determined from the flows of the separate channels as well as from the δD values of the water. See Figure 4 for map*

Date	Flow channel	Flow(1) (m^3/sec)		Proportions (%) of Loisach and Loisach-Isar-Kanal water in Isar water after confluence	
				Calculated from Flow	δD values
9 October 1968	Isar	27.0		26	30 ± 5
	Loisach	43.0			
	Loisach-Isar-Kanal	33.5		74	70 ± 5
8 November 1968	Isar	19.5		42	37 ± 5
	Loisach	23.0	26.5		
	Loisach-Isar-Kanal	3.5		58	63 ± 5
	Isar	19.5		85	78 ± 5
	Loisach-Isar-Kanal	3.5		15	22 ± 5

1. According to data furnished by the Bavarian Land Office for Hydrology, Munich. We wish specially to thank ORBR Dr.-Ing. Streil for having made this information available to us.

given in each case. The straight line plotted reveals a variation of the D content of $\Delta\delta D = -3.5$ per thousand per 100 m rise in elevation. This corresponds, more or less, to the value found for the isotope altitude effect for precipitation in the central mountains of the Federal Republic of Germany.

Based on the example of the Loisach, Figure 3 shows the change, measured on 8 November 1968, in the D content along the stream section due to inflows and to its passage through lakes. At the confluence of two rivers, it is possible to calculate from such D balances the proportion of the separate flows, provided the δD values differ sufficiently. Table 1 shows, for the confluence near Wolfratshausen of the Loisach, the Loisach-Isar-Kanal and the Isar (Figure 4), that the flow proportions so determined agree fairly well, within the limits of experimental error, with measured values.

With respect to the lakes, the wide variations of the δD values between inflow and outflow cannot generally be explained by mingling effects alone, since, owing to the long residence time of the water, D enrichment may take place as a result of evaporation from the lake surface. In the present case, too few measurements are available for a quantitative water balance of lakes to be obtained form isotope measurements as in Zimmerman (1969). Nevertheless, Figure 1 reveals that the lakes with a large surface area and longer residence times (Ammersee, for example), the difference in δD values between the inflow and outflow is greater than for smaller lakes that are traversed more rapidly (Kochelsee, for instance). The difference between the δD values of the separate measuring series necessitate further work; since measurements are available only for November, February and June, this information is incomplete and must be augmented by measurements embracing several complete years.

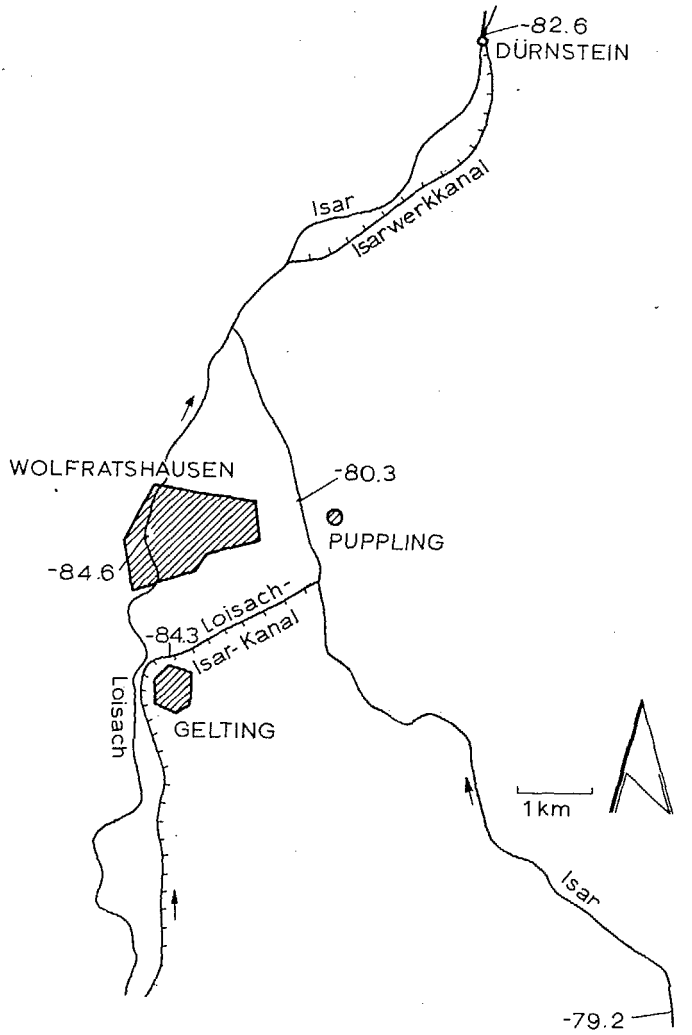


Figure 4 – *Sampling points in the confluence area of the Loisach and Loisach-Isar-Kanal with the Isar*
The figures indicated are δD values and apply to samples gathered on 9 October and 8 November 1968

Figure 5 presents the variation with time of the D content of several characteristic sampling points; good reproduction is obtained of the November 1968 and November 1969 values for all surface waters examined.

Measurements on the Loisach and Partnach flowing out of the high mountains clearly show a drop in the June observations as a result of the snowmelt in early summer. Underlying the increase of the δD value at the end of February is the fact that, during the winter months, water flows only from low-altitude regions. Although the δD value for the outflow from the Murnauer Moos has become fairly constant in time, ther still remains an indication of the pattern characteristic for the Loisach.

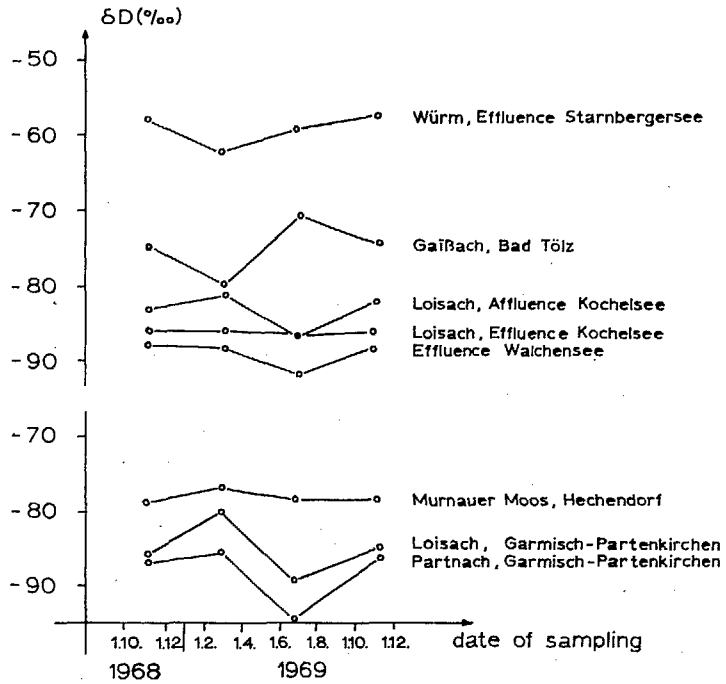


Figure 5 – *Variation with time of the δD values of surface waters in the southern catchment area of the Isar. Sampling on 8 November 1968, 27 February 1969, 23 June 1969 and 21 November 1969.*

The values obtained at Kochelsee give evidence of the mixing that occurs during the flow across this body of water. The annual means of the δD values of the two discharges furnish the constant δD value for the effluent.

The δD values for the Gaissach, coming from a low-altitude region with only a small difference in elevation and joining the Isar near Bad Tölz, show a minimum at the end of February and a maximum in June which corresponds to the pattern of the D content in precipitation (Dansgaard, 1964). This implies that the residence time between precipitation and runoff of the water of this channel is short.

The fluctuations of the δD value for the Würm flowing out of Starnbergersee are very likely a function of enrichment due to evaporation from the relatively large lake surface, which is more pronounced in summer and autumn than in spring. However, rain water running off directly may provide an additional factor to be taken into account.

This method for calculating flow proportions from the δD values can also be employed with the annual curve of the δD value, for a quantitative determination of the flows from different watersheds. For the Isar at Wolfratshausen the proportions of alpine water and foothill water were ascertained from the δD value for the Isar at Lenggries, fed almost exclusively by water from the Karwendel Range, and from the average of the δD values for the tributary creeks running in from the foothills between Lenggries and Wolfratshausen. The periods investigated were in November 1968 and June 1969 (Table 2). The measured values, together with earlier values for December 1963 and May 1964 (Eichler, 1964), provide a good picture of the annual fluctuations.

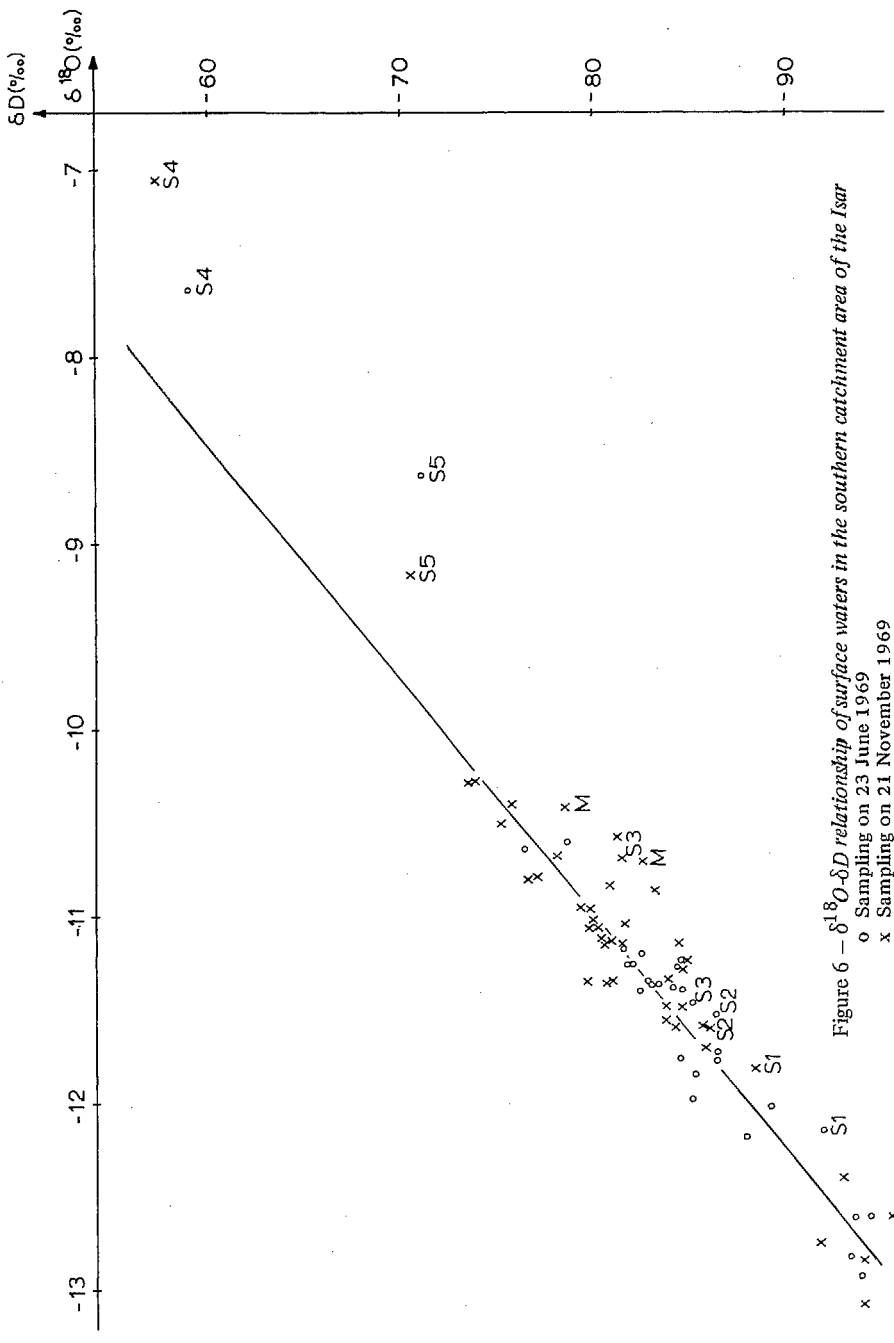


Figure 6 – $\delta^{18}O$-δD *relationship of surface waters in the southern catchment area of the Isar*
o Sampling on 23 June 1969
x Sampling on 21 November 1969
S Lake effluents (S 1, Walchensee ; S 2, Kochelsee ; S 3, Sylvenstein-Stausee ; S 4, Starnbergersee ; S 5, Osterseen)
M Murnauer Moos

Table 2. *Proportions of alpine and foothill water in the Isar at Wolfratshausen.*

Sampling date	Flow proportions	
	Alpine water	Foothill water
8/9 November 1968	77 ± 5	23 ± 5
December 1963	90 ± 10	10 ± 10
May 1964	67 ± 10	33 ± 10
23 June 1969	70 ± 10	30 ± 10

The relationship represented in Figure 6 between the D content and the ^{18}O content shows that, for the waters examined, normal values of m and t are generally obtained.

The series of measurements in Figure 6 show that between November and June a shift occurs in the average δD and $\delta^{18}O$ values towards the negative. This signifies that right into early summer a large part of the streams are fed by snowmelt.

The measurement values for Osterseen and Starnbergersee do not fall within the range of normal values of equation 2. It must be assumed that in these lakes kinetic isotope effects take place due to rapid evaporation. Compared with the other alpine lakes investigated (Walchensee, Kochelsee and Sylvensteinstausee), this is the result of the warmer climate and longer exposure to sunshine in the lowland, the larger lake surface and the greater residence time of the water.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Brunnée, C.; Voshage, H. 1964. Massenspektrometrie. München.

Craig, H. 1957. Isotopic standards for carbon and oxygen and correction factors for mass-spectrometric analysis of carbon dioxide. *Geochim. Cosmochim. Acta,* vol. 12, p. 133-40.

—, 1961. Standard for reporting concentrations of deuterium and oxygen-18 in natural waters. *Science,* vol. 133, p. 1833-4.

Dansgaard, W. 1961. The isotopic composition of natural waters with special reference to the Greenland ice cap. *Medd. Groenland,* vol. 165 p. 1-120.

—, 1964. Stable isotopes in precipitation. *Tellus,* vol. 16, p. 436-68.

Eichler, R. 1964. Uber den Isotopengehalt des Wassers in Niederschlags-, Boden- und Grundwasser. Dissertation Universität Bonn.

Friedman, I. 1955. Deuterium content of natural waters and other substances. *Geochim. Cosmochim. Acta,* vol. 4, p. 89-103.

—, Redfield, A. C.; Schoen, B.; Harris, J. 1964. The variation of the deuterium content of natural waters in the hydrologic cycle. *Rev. Geophys.,* vol. 2, p. 177-224.

Guidebook on Nuclear Techniques in Hydrology, 1968, Wien, IAEA.

Horibe, Y.; Ogura, N. 1968. Deuterium content as a parameter of water mass in the ocean. *J. Geophys. Res.,* vol. 73, p. 1239-49.

Moser, H.; Stichler, W. 1970. Deuterium measurements on snow samples from the Alps. *Symposium on Use of Isotopes in Hydrology, IAEA,* Wien, 1970, paper SM-129/4.

Zimmermann, U. 1969. Bilanzierung von Seen mit Hilfe von stabilen Isotopen. Dissertation Universität Heidelberg.

DISCUSSION

Vugts: 1. Why do you not have a double collector system consisting of two separate collectors side by side for measuring CO_2-^{18}O as no corrections of the ratio would be necessary?

2. Why did you not use O_2, then you can measure the 34-32 ratio, which is more accurate than CO_2 measurements?

Stichler: The first question concerned the structure of the mass-spectrometer. We are using a spectrometer (ATLAS, Bremen) which measures masses 2 and 3; the relation of hydrogen isotopes is, therefore, clear. For measurement of oxygen, we use CO_2 and let the masses pass through a slit on a plate, first of all mass 46, then 44 and then 48. Masses 38 or lower and masses above 48 are not detected.

Vugts: And mass 40 of argon?

Stichler: We are using a test gas CO_2 with argon. As to your second question, I should like to know a method for getting the oxygen contents quantitatively. I should also prefer to measure masses 32 and 34.

Fröhlich: I think that water contains much information which can only be discovered by the appropriate physical methods. This applies to ^{18}O, deuterium and also to tritium. We have found in the course of our investigations on the amount of surface and ground-water in the water of the River Havel that it is better to make complex tests. My questions:

1. Is it also your opinion that it is preferable to use stable isotopes as well as tritium? What advantages do you see in applying a complex method?

2. Your investigations have been made in the Alps, a region with very varying physical conditions, leading to the differentiation of isotopes. We have investigated water in the lowland or in low-mountainous areas where these strong differences do not exist. I should like to know whether, under these conditions, stable isotopes would also promise such clear results, or, to generalize the question, do viewpoints exist on whether the application of stable isotopes is specific for certain hydrological or hydrometric problems?

Moser: 1. It is always advantageous to measure deuterium *and* tritium. The variation of the contents of deuterium or tritium is subject to specific physical laws. The annual variation of deuterium used to double, whereas the content of tritium is decreasing now atmospheric testing of bombs has ceased.

2. The changes of deuterium and ^{18}O are mainly caused by changes of the phase and the temperature. This leads to differences in the contents of deuterium due to climatic changes, melting of ice or evaporation from the surface of lakes. Measurements of deuterium or ^{18}O, therefore, may lead to results in areas without orographic data.

Martinec: 1. Is the coincidence between the yearly pattern of isotopic composition of precipitation and discharge a conclusive proof of the short residence time of water in the watershed?

2. I would like to know whether the enrichment in stable isotopes of snow cover during the winter is mainly caused by fractionation or by subsequent rainfalls of heavier isotopic composition.

Stichler: 1. A combined measurement of the natural distribution of tritium, deuterium and ^{18}O in water has already been recommended by Mr. Moser. However, a measurement of the annual variation of the content of deuterium in the water sample

in comparison with water from precipitation may give some idea of the time the water has spent in the soil.

2. Normally there is no duplication of efforts. In our experience, the changes of the content of deuterium are important with regard to the ageing of snow.

Utilisation des tracteurs radio-actifs pour la mesure des débits de charriage et pour celle des matériaux en suspension

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Résumé. En sédimentologie les traceurs radio-actifs sont employés principalement pour mesurer les débits de charriage et la teneur des eaux en matériaux en suspension. Les principes d'utilisation des traceurs et les procédés de réalisation (irradiation en réacteur nucléaire de verre broyé ou marquage du sédiment naturel par dépôt superficiel ou absorption d'un sel ou d'un métal radio-actif) sont passés en revue.

Charriage. La discussion des méthodes d'injection, de détection du traceur ainsi que l'interprétation des résultats sont à la base d'une estimation quantitative du débit de charriage appelée méthode du bilan des taux de comptage où l'intégrale des taux de comptage mesurés par un détecteur parcourant le nuage radio-actif est reliée à l'épaisseur moyenne de sédiment en mouvement. La critique de cette méthode est faite à l'aide d'exemples de son application en rivière et en mer où des épaisseurs de 30 cm ont été mesurées et vérifiées par carottage.

Matériaux en suspension. Les deux principales techniques de mesure en continu des matériaux en suspension se fondent l'une sur la mesure de l'atténuation d'un rayonnement nucléaire due à ces matériaux, l'autre sur la variation de la radio-activité naturelle du milieu avec la teneur des matériaux en suspension. Des exemples comparés illustrent ces méthodes et leurs possibilités.

RADIOACTIVE TRACERS APPLIED TO MEASUREMENTS OF SUSPENDED LOAD AND BEDLOAD DISCHARGE

Abstract. In sediment transport, radioactive tracers are mainly used to measure bedload discharge and suspended sediment concentrations. The paper presents the principles of utilization of tracers and methods of preparation (irradiation of ground glass in nuclear reactors or labelling of the natural sediment either by surface deposit or by adsorption of a radioactive chemical or metal).

Bedload movement. A quantitative estimate of bedload movement is possible. Proper injection and detection of the tracer cloud and discussion of results allow a theory of interpretation called "The Count Rate Balance", where the mean thickness of the "moving sediment" is related to the balance of the counting rates measured by a detector scanning the tracer cloud. This method has been used in rivers and on coasts where thicknesses of up to 30 cm have been measured and checked by coring.

Suspended load. The two main techniques of continuously measuring the concentration of suspended sediment are based either on attenuation or backscattering of nuclear radiation due to suspended sediments or on the natural radioactivity of these sediments. Some examples show the comparison between both methods and their potentialities.

TRAZADORES RADIACTIVOS APLICADOS A LAS MEDICIONES DE SEDIMENTO EN SUSPENSIÓN Y ARRASTRE DE FONDO

Resumen. Para el estudio de sedimentos, se utilizan trazadores radiactivos, principalmente para medir el arrastre de fondo y la concentración de sedimentos en suspensión. Esta comunicación presenta los principios de utilización de trazadores y métodos de preparación (irradiación de cristal base en reactores nucleares o marcado de sedimentos naturales por depósito superficial o por absorción de compuestos o metales radiactivos).

Movimiento del arrastre de fondo. Es posible una estimación del movimiento del arrastre de fondo. La inyección y detección de la nube de trazador y los resultados que se obtienen permiten una teoría de interpretación llamada "the count rate balance" en la cual el espesor medio del "sedimento en movimiento" se relaciona con el equilibrio de los valores medidos por un detector

aplicado a la nube. Este método se ha utilizado en ríos y costas en los que se han medido y comprobado espesores de 30 cm.

Sedimento en suspensión. Las dos principales técnicas para medir la concentración de sedimento en suspensión se basan en la atenuación o en la retrodispersión de una radiación nuclear debida a sedimentos suspendidos o a la radiactividad natural de dichos sedimentos. En algunos ejemplos se comparan ambos métodos y sus posibilidades.

ПРИМЕНЕНИЕ РАДИОАКТИВНЫХ ИНДИКАТОРОВ ДЛЯ ИЗМЕРЕНИЯ ВЗВЕШЕННЫХ И ДОННЫХ НАНОСОВ

Резюме. В транспорте наносов радиоактивные индикаторов используются главным образом для того, чтобы измерить расход донных наносов и концентрацию взвешенных наносов.

Доклад излагает принципы использования индикаторов и методы приготовления (иррадиация матового стекла в атомных реакторах или мечение естственных наносов, путем либо поверхностного отложения или поглощением радиоактивных химикалиев или металла).

Движение донных наносов. Количественная оценка движения донных наносов является возможной. Надлежащий ввод и обнаружение индикаторного облака и анализ результатов допускают теорию интерпретации, названную « the count rate balance », где средняя толщина слоя движущихся наносов связана с балансом вычисленных величин, измеренных детектором, развертывающим индикаторное облако.

Этот метод использовался на реках и побережьях, где толщины измеряемых слоев достигали 30 см и проверялись затем путем взятия пробы.

Взвешенные наносы. Два главных технических приема непрерывного измерения концентрации взвешенных наносов основываются на ослаблении или рассеянии ядерной радиации, вызываемой взвешенными наносами, или на естественной радиоактивности этих наносов. Некоторые примеры показывают сравнение между обоими методами и их возможности.

INTRODUCTION

En sédimentologie les radio-éléments trouvent emploi dans deux directions principales : (a) l'étude de la dynamique des sédiments en milieu fluvial et maritime ; (b) la mesure en continu de la quantité de sédiments en suspension.

Nombreux sont les problèmes liés au génie civil où les traceurs peuvent fournir des résultats spectaculaires ; ce sont les études d'efficacité de dragage, les opérations de protection des côtes et de l'entrée des chenaux d'accès, les études systématiques de sites importants, la mesure de l'apport solide dans un réservoir, les études des bancs de sable en rivière navigable, le tarage des modèles réduits – et cette liste n'est nullement exhaustive.

Les premières expériences dans le monde datent de 1954-1955, où simultanément au Royaume-Uni et en France, puis au Japon et au Portugal, des études en estuaire ou sur des plages étaient entreprises (Gibert, 1955; Hours *et al.*, 1955; Inose *et al.*, 1955; Kidson *et al.*, 1956; Putman *et al.*, 1954). Depuis, une bonne vingtaine de pays ont réalisé des expériences semblables, mais seulement deux ou trois pays emploient les traceurs de façon presque courante.

LES METHODOLOGIES

Les méthodes utilisées sont de deux types, suivant que l'on s'intéresse à la dynamique des sédiments ou à la mesure en continu de leur concentration. Dans le premier cas, un traceur artificiel radio-actif est injecté dans le milieu à étudier et sa migration est suivie à l'aide de détecteurs de rayonnement. Dans le deuxième cas, la variation du rayonnement reçu par un détecteur est reliée à la variation de concentration dans le milieu où il est plongé. Cette méthodologie sera étudiée en même temps que son application pratique.

La dynamique sédimentaire et la méthodologie classique

Pour commencer, rappelons l'existence des traceurs naturels ou traceurs de l'environnement, ils peuvent être de trois types : (a) les radio-éléments existant naturellement dans les sédiments ; (b) les retombées radio-actives qui se fixent facilement sur les sédiments ; (c) les rejets des usines et des réacteurs nucléaires.

Leur recherche s'apparente aux méthodes pétrographiques (minéraux lourds, débris coquilliers), qui permettent de remonter à l'origine des matériaux et, par suite, de déterminer la direction de transport (Byerly, 1962; Kaufman et Klingemann, 1961; Rivière et Vemhet, 1966).

Le choix du traceur, son activité, sa masse

Lorsqu'on ne recherche pas de résultats quantitatifs, de très nombreux traceurs peuvent être utilisés. Une liste des caractéristiques d'expériences mettrait en évidence l'emploi d'une douzaine d'émetteurs différents, auxquels il conviendrait d'en ajouter d'autres tels le ^{115}Cd, ^{86}Rb, ^{95}(Zr + Nb), ^{58}Co utilisés sur le terrain, le ^{24}Na, ^{122}Sb utilisés en modèle, le ^{82}Br, ^{131}I utilisés pour des mouvements de vases. Tous ces traceurs sont des émetteurs γ détectables sur le terrain, seul le ^{32}P, émetteur β pur, nécessite le prélèvement d'échantillons. Le ^{51}Cr et le ^{46}Sc sont les plus utilisés dans le monde. Le choix du traceur est guidé par sa période et son rayonnement. La période doit être prise dans l'ordre de grandeur du phénomène étudié ; elle dépend aussi de la fréquence désirée de répétition de l'expérience. En fait, c'est la dispersion qui limite la durée plus que la période du traceur. Un émetteur γ dur est nécessaire si l'on croit à un enfouissement important. En fait, le choix est souvent une question d'habitude : ^{51}Cr en République fédérale d'Allemagne, ^{46}Sc aux Pays-Bas, ^{192}Ir en France.

Les activités utilisées varient énormément suivant les expériences : de quelques millicuries efficaces jusqu'à une centaine de curies. Le choix de cette activité dépend des vitesses de transport, de la dispersion, de l'enfouissement, etc. S'il n'est pas possible de déterminer l'activité totale qu'il faut injecter, il est par contre possible de déterminer l'activité spécifique ou rapport activité sur masse de traceur pour un détecteur donné, un mode de détection donné, un sédiment donné et des conditions de précision choisies (Courtois et Sauzay, 1970 a). Le principe du calcul repose sur la distribution statistique des grains de traceurs autour du détecteur ; le nombre de grains "vus" par le détecteur doit être tel qu'en limite du nuage radio-actif le taux de comptage soit à peu près indépendant de la distribution géométrique des grains.

La réalisation du traceur

Irradiation en réacteur nucléaire de sédiment naturel. On obtient généralement des émetteurs de courte période : ^{31}Si, ^{24}Na. Toutefois, on a mentionné dans certains sables du phosphore, du fer, du scandium, mais en faibles quantités.

Marquage massique par incorporation d'un isotope radio-actif dans la masse d'un simulateur de sédiment. Les deux principaux simulateurs sont le verre et une résine échangeuse.

La zéolite (ionac ^{50}C ou Green Sand) est une résine qui absorbe facilement le ^{46}Sc. Elle doit être chauffée vers 800-900 °C pour acquérir une dureté suffisante. Le marquage est massique, c'est-à-dire que l'activité est proportionnelle à la masse.

Quant au verre, on le broie avant irradiation, après y avoir incorporé le traceur. C'est le procédé le plus courant.

Dépôt en surface du sédiment naturel d'un élément radio-actif. De nombreuses méthodes s'affrontent et il est impossible de tout reproduire ici. On se reportera à Courtois (1966). Ce qu'il faut surtout obtenir c'est un rendement de fixation élevé et reproductible, ainsi qu'une bonne solidité du dépôt.

Cas particulier des vases et des galets. Pour le marquage des vases, deux théories s'affrontent : le verre est-il un bon simulateur des vases ? L'adsorption directe de traceur par la vase change-t-elle le comportement dynamique des vases ? Sur la base de nos propres expériences nous repondrions plutôt par la négative à chacune des deux questions. A l'heure actuelle on ne peut encore leur donner de réponse définitive.

En ce qui concerne le marquage des galets, il est obtenu par mise en place d'un morceau de fil ou d'une bille radio-active dans un trou creusé dans chaque galet.

LA DYNAMIQUE SEDIMENTAIRE ET L'ETUDE DE CHARRIAGE

Il n'est pas possible de parvenir à la mesure du débit instantané du charriage, mais on obtient celle du transport effectué entre deux détections successives, c'est-à-dire le débit moyen de charriage entre deux dates. On détecte donc un mélange sable-traceur qui n'est que très lentement en mouvement lors de la détection. L'épaisseur de ce mélange est inconnue avant l'expérience et il est donc impossible de réaliser une injection du traceur sur une épaisseur correspondant à ce mélange. Le traceur est donc déposé à la surface du lit en taches ponctuelles ou linéaires. Les systèmes d'injection sont très variés et reflètent l'imagination de leurs auteurs. Après l'immersion du traceur se pose le problème de l'obtention du "bon mélange", autrement dit un mélange du traceur et du matériau de fond tel que, lorsqu'il est obtenu, les valeurs relatives au mouvement du fond (déduites des mesures du mouvement du traceur) ne dépendent plus des conditions initiales, c'est-à-dire ne dépendent plus des conditions d'immersion. Cela est quasiment impossible à réaliser et l'on travaille toujours sans que le "bon mélange" soit réalisé. Il est difficile de chiffrer cet effet, mais l'expérience personnelle permet de dire quand il semble être négligeable.

Calcul du débit de charriage (vitesse et épaisseur)

La seule méthode pratiquement utilisée est celle de l'injection instantanée, en général ponctuelle ou multiponctuelle, plus rarement linéaire. La méthode d'injection continue a été réalisée sur les plages (Russel, 1961). En hydrologie, on réalise l'intégrale des taux de comptage dans le temps et l'on a

$$Q \int n\, dt = A \times F \quad \text{où} \quad \begin{array}{l} Q = \text{Débit} \\ A = \text{Activité} \\ F = \text{Facteur d'étalonnage} \end{array}$$

En sédimentologie, ce n'est pas possible et l'on effectue une détection dans l'espace durant un temps Δt considéré comme petit dans l'échelle de temps du mouvement de charriage. Une carte des taux de comptage à l'instant t (jour ou heure habituellement) peut être dressée. La variation dans le temps de la position centre de gravité de ces taux de comptage est reliée à la "vitesse moyenne" de charriage. Etant donné que le transport du sédiment se fait à la surface du lit, cette vitesse moyenne du mélange n'est qu'une vitesse apparente ; elle est parfois appelée vitesse débitante. Deux effets peuvent perturber la détermination de cette vitesse.

En premier lieu, la détection est limitée aux zones où le taux de comptage est significativement différent du bruit de fond et certaines zones à très faible activité ne seront pas considérées (effet de seuil).

En second lieu, du fait de l'absorption du rayonnement γ par la matière, les grains enfouis donnent une information plus faible que les grains en surface, ils n'auront donc pas le même poids dans le calcul du centre de gravité. Le centre de gravité des taux de comptage n'est pas celui des particules de traceur (effet d'enfouissement).

Il est beaucoup plus difficile de déterminer E, "épaisseur" de charriage, c'est-à-dire l'épaisseur du lit qui a la vitesse apparente V_m, ou encore l'épaisseur de mélange du traceur.

Le débit volumique de charriage sera alors calculé par

$$Q = V_m \times E_m$$

ou débit volumique par unité de longueur perpendiculaire à la direction du transport.

On peut obtenir l'épaisseur du mélange par carottage, mais, afin d'avoir une bonne précision, il faut énormément de grains de traceur (activité spécifique faible), et le carottage n'est pas chose aisée.

Cette épaisseur peut être déterminée par des considérations géométriques et statistiques sur l'amplitude des hauteurs des rides et des dunes.

On peut aussi injecter un nombre très limité de grains qu'on détectera individuellement (Courtois, 1966).

Un autre moyen consiste à appliquer la méthode du bilan des taux de comptage (Sauzay, 1968) où l'intégrale dans l'espace des taux de comptage N à un instant t est reliée à l'épaisseur du mélange E. Cela suppose la connaissance de la répartition en profondeur de la valeur de la vitesse apparente $V(z)$ ainsi qu'un étalonnage de la sonde détectrice $f(z)$.

$$N = F\,[E\,,A\,,V(z)\,,f(z)]$$

A est l'activité totale injectée.

Le grand intérêt de cette méthode réside dans le fait qu'elle n'est qu'un puissant moyen d'interprétation, d'une détection faite selon les procédés classiques. Plus cette détection sera soignée et plus l'interpréation par cette méthode sera détaillée.

Le principe de cette méthode est le suivant : d'autant plus grande est l'épaisseur de mélange d'une activité de traceur, d'autant plus faible est le taux de comptage reçu par un détecteur situé en surface. En chaque point du lit, il existe deux inconnues : l'activité détectée *"a"* et son enfouissement *"e"*. L'utilisation simultanée de deux détecteurs à coefficients d'étalonnage différents (un seul détecteur à deux sorties réglées sur deux seuils différents et travaillant en discriminateur pourrait convenir) devrait fournir les deux valeurs *"a"* et *"e"*. Une étude statistique des

valeurs de *"e"* donnerait la valeur moyenne E cherchée. On pourrait aussi apporter une correction pour l'effet d'enfouissement vu précédemment. Mais il est très difficile d'obtenir des coefficients d'étalonnage suffisamment différents ; aussi, intègre-t-on sur tout le nuage et l'intégrale de l'activité *"a"* est l'activité A injectée. Par suite de l'intégration, la valeur de l'épaisseur se transforme en une valeur moyenne E qu'on utilise pour calculer le débit de charriage. Cette moyenne n'est pas la moyenne arithmétique sur le nuage de l'épaisseur en chaque point, mais la valeur maximale. On obtient ainsi une valeur par excès du débit. Cela explique probablement qu'en certains cas, on n'ait pas concordance entre cette estimation et un carottage important, alors que de nombreuses expériences ont montré un bon accord.

Ce procédé est systématiquement appliqué en France dans toutes les expériences menées par la Section d'application des radio-éléments du Commissariat à l'énergie atomique.

Cette méthode de dépouillement des résultats de détection combinée aux travaux sur la statistique du nombre de grains (Courtois, 1970) permet une étude détaillée des différentes zones de transport couvertes par le nuage radio-actif. Ce qui veut dire qu'une interprétation de la façon dont le transport se fait peut aussi être donnée.

Pour une discussion plus approfondie des détails de ces méthodes, on se mettra en contact avec les auteurs ou l'on se reportera à la bibliographie publiée par Courtois en 1966 (Courtois, 1966).

Exemples d'application

Stung-Sen, affluent du Tonlé-Sap (République khmère). Le charriage de fond se fait avec une épaisseur de mélange d'environ 10 à 15 cm déterminée par carottage, par la méthode du nombre de grains limités (Courtois, 1966) et par la méthode du bilan des taux de comptage (Anguenot *et al.,* 1964).

Sienne. Fleuve côtier de la Manche (France). L'épaisseur de mélange est de 12 cm et c'est la seule mesure effectuée (Giresse, 1966).

Cher (Sauzay, 1968). Suivie sur plusieurs détections, l'épaisseur de mélange a augmenté de 6 à 12 cm, puis est restée constante alors que la vitesse moyenne du nuage décroissait : cela est bien en accord avec la variation de la capacité de transport estimée par les formules de Meyer Peter et de Kalinske. Lors de la première méthode, le flot augmente, entraînant une augmentation du charriage, qui se traduit par une augmentation de l'épaisseur de mélange. Dans la deuxième période, le flot décroit régulièrement, l'épaisseur de mélange reste la même, mais le centre de gravité du traceur se déplace de moins en moins vite. Les débits de charriage ont varié entre 95 ± 20 et 30 ± 4 t/j pour la fraction granulométrique inférieure à 1 mm, la seule observée. L'erreur correspond à une déviation standard et non à l'erreur probable.

Côte du Roussillon, mer Méditerranée (France). Plusieurs expériences de transport côtier sous l'effet de la houle, des vents et des courants ont été faites. La méthode du bilan des taux de comptage a donné des épaisseurs variant entre 12,5 et 19,5 cm en différents points, quand les carottages donnaient 15 et 17 ou 20 cm. Le débit était de 2 à 2,3 t/j ce qui se compare assez bien avec l'estimation théorique déduite du bilan énergétique de la houle : 2,7 t/j.

Lac de Maracaïbo (Venezuela). Là aussi on a trouvé des résultats concordant bien, sauf dans un cas, où les carottes ont montré un "recouvrement" pur et simple du nuage radio-actif par des sédiments de granulométrie plus fine. Le centre de gravité

n'avait pas bougé, montrant l'absence de mouvement du traceur. La méthode du bilan telle qu'elle est établie ne s'applique qu'aux zones de mouvement et non de sédimentation. En fait, cela est valable pour toute expérience de traceurs.

Etudes de vases et de galets. Les vases peuvent donner lieu à deux types d'expériences : les études de diffusion en suspension dans un estuaire et les études de mouvements au fond de fortes concentrations. Le premier cas est un problème de lois de diffusion ; dans le deuxième, les méthodes précédentes peuvent être appliquées avec une technologie de détection particulière.

Les galets donnent lieu à des détections individuelles où la radio-activité donne l'enfouissement individuel des grains (méthode du nombre de grains limités). Des considérations de distributions statistiques de la position des grains permettront d'évaluer le centre de gravité, l'épaisseur de mélange, etc. Le problème est assez délicat, du fait de l'hétérogénéité des cours des rivières charriant des galets.

Conclusions sur le charriage

De nombreuses autres expériences pourraient être citées montant que ces travaux par traceurs requièrent actuellement une certaine expérience et un peu de sens pratique de la part de l'expérimentateur. Chaque site sera l'objet d'un cas d'espèce et ces méthodes d'interprétation doivent être considérées plus comme un guide, une ligne de conduite pour le dépouillement et l'interprétation des résultats que comme un outil affiné travaillant avec une précision connue. Néanmoins, ces méthodes ont fait leurs preuves et les résultats obtenus sont nombreux et valables.

LA MESURE EN CONTINU DES SEDIMENTS EN SUSPENSION

Deux méthodes s'opposent: l'une, toute récente, est basée sur l'étude du rayonnement naturel des sédiments et est appliquée avec succès dans des estuaires marins ; l'autre, plus ancienne, est basée sur l'absorption ou la diffusion d'un rayonnement $\gamma(^{241}\text{Am})$ ou $X(^{109}\text{Cd})$ par les sédiments en suspension. Ce dernier type correspond à une véritable mesure de la densité relative de l'eau chargée par rapport à l'eau pure.

Jauges de densité (Courtois *et al.*, 1970 b; Florkowski et Cameron, 1965; Florkowski 1970; McHenry *et al.*, 1966 ; Ziegler *et al.*, 1967).

Les jauges à ^{109}Cd permettent une mesure ponctuelle de la concentration. Le rayonnement X est absorbé par la matière située entre la source et le détecteur.

Les jauges à ^{241}Am permettent soit une mesure ponctuelle (absorptiométrie), soit une mesure moyenne (diffusion) de la concentration dans un volume d'environ un quart de mètre cube.

Les jauges sont sensibles aux variations de géométrie, légèrement à la salinité et peuvent, suivant les conditions d'application, être sensibles à la composition chimique du sédiment. Ce facteur semble être plus important dans des eaux de fleuves tropicaux que dans les estuaires des régions tempérées ou tropicales. La stabilité électronique est importante pour des appareils autonomes.

L'erreur résultante semble être de 400 à 600 mg/litre. Cependant, après une étude très détaillée, nous pensons qu'elle serait proche de 1 g/litre.

Les concentrations mesurables par ces appareils sont supérieurs à 1 g/litre.

Jauge à radio-activité naturelle. Les sédiments et principalement les vases sont naturellement radio-actifs. Le rayonnement mesuré est donc relié à la quantité de sédiments en suspension autour du détecteur. Le signal reçu par le détecteur est donc fonction de la turbidité, mais aussi du bruit de fond cosmique ou provenant du lit et des sels dissous (pratiquement le ^{40}K du ClK). Les coefficients de la formulation générale reliant la turbidité, la salinité, le bruit de fond, etc., ont été mesurés pour des vases de différentes origines et montrent des variations peu importantes indiquant une faible influence de la composition chimique des vases. Ainsi, moyennant un étalonnage préalable, ce système pourrait être utilisé pour tout problème de vases en suspension.

Cette mesure est surtout sensible à la salinité, qu'il importe de bien mesurer, à la position de la sonde par rapport au fond et à la stabilité électronique. L'erreur de mesure serait de 600 mg/litre aux environs de 10 g/litre Les performances sont à peu près les mêmes : concentrations supérieures à 1 g/litre. Cette méthode n'a été testée qu'en estuaire marin.

Conclusion sur la mesure des sédiments en suspension

Différents procédés permettent de mesurer des concentrations supérieures à 1 g/litre par des comptages relativement brefs en temps. Des appareils à autonomie prolongée existent, mais il faut toujours prendre soin des problèmes de dérive électronique et de l'effet de la composition chimique.

Les concentrations inférieures à 1 g/litre ne peuvent pas dans l'état actuel des connaissances et des réalisations être mesurées par ces méthodes.

BIBLIOGRAPHIE / BIBLIOGRAPHY

Alquier, M.; Courtois, G.; Gruat, J.; Sauzay, G. 1970. La notion de "bon mélange" dans l'emploi de traceurs. Vitesse débitante et vitesse du centre de gravité d'un nuage de traceurs dans l'étude d'un écoulement. *Use of isotopes in hydrology.* Wien, IAEA.

Anguenot, F.; Carbonnel, J.P.; Courtois, G.; Danion, J.; Forsberg, H. 1964. *Etude du charriage du Stung-Sen au Cambodge.* (IAEA technical assistance report n° 10, Wien).

Byerly, J.R. 1962. *The relationship between watershed geology and beach radio-activity.* Berkeley, University of California, Institute of Engineering Research, Hydraulics Engineering. Laboratory, Wave Research, Project HEL-43.

Courtois, G.; Hours, R. 1965. Propositions concernant les conditions particulières d'emploi des radio-éléments artificiels pour étudier les mouvements de sédiments, *EURISOTOP. Cahier d'information,* n° 8. (Communication n° 29, p. 441-480 ; Rapport interne SAR-S (64-13.)

Courtois, G. 1966. Emploi des radio-éléments en sédimentologie. *Use of isotopes in hydrology.* Wien, IAEA.

Courtois, G.; Sauzay, G. 1970. Sur les masses de sédiments à injecter dans une expérience de traceurs radio-actifs en sédimentologie dynamique. *Use of isotopes in hydrology.* Wien, IAEA.

Courtois, G.; Anguenot, F.; Magloire, C. 1970. Turbidimétrie X-γ ou radio-activité naturelle ? *Use of isotopes in hydrology.* Wien, IAEA.

Florkowski, T.; Cameron, J.F. 1965. A simple radioisotope X-ray transmission gauge for measuring suspended sediment concentration in rivers. *Radioisotopes instruments in industry and geophysics.* Wien, IAEA.

Gibert, A. 1955. *Essais sur la possibilité d'employer le ^{110}Ag dans l'étude du transport du sable de la mer.* Lisboa, Minesterio das Obras Publicas, Laboratorio Nacional de Engenharra Civil. (Publication n° 63).

Giresse, P. 1966. Contribution des traceurs radio-actifs à l'étude de la dynamique des dépôts de l'estuaire de la Sienne. *Cahiers océanographiques (COEC),* vol. 18, n° 1, p. 1-8, janvier 1966.

Hours, R.; Nesteroff, W.D.; Romanvosky, V. 1955. Méthode d'étude de l'évolution des plages par traceurs radio-actifs. *Travaux du Centre de recherches et d'étude océanographiques* (1, quai Branly, Paris), vol. 1, n° 11, novembre 1955.

Inose, S.; Kato, M.; Sato, S.; Shiraishi, N. 1955. *The field experiment of littoral drift using radioactive glass sand.* (Communication P 1053.) [Conférence de Genève.]

Kidson, D.; Smith, D. B.; Steers, J. A. 1956. Drift experiments with radioactive pebbles. *Nature* vol. 178, n° 4257.

Klingemann, P.C.; Kaufman, W.J. Transport of radionuclides with San Francisco Bay sediment. *Progress report 1961-1962.* Berkeley, University of California.(SERL. 63/7).

McHenry, R.J.; Neil, L.C.; Willis, J.C.; Murphree, C.E.; Bolton, G.C.; Sanson, O.W.; Gill, A.C. 1966. Performance of nuclear sediment concentration gauges. *Use of isotopes in hydrology*. Wien, IAEA.

Putman, J.L.; Smith, D.B.; Wells, H.M.; Allen, F.; Rowan, G. 1954. *Thames siltation investigation. Preliminary experiments of the use of radioactive tracers for indicating mud movements.* (Rapport AERE/I/R, 1576 ; Harwell, déc. 1954).

Rivière, A.; Vernhet, S. 1966. Signification sédimentologique de la radio-activité naturelle des plages du golfe du Lion, *C.R. Acad. des sciences,* 262, Série D, 24 janvier 1966, p. 440-443.

Russel, R. 1961. The use of fluorescent tracers for the measurement of littoral drift. *Proceedings of the 7th Conf. on Coastal Engineering,* vol. 1, p. 413-444.

Sauzay, G. 1968. *Méthode du bilan des taux de comptage d'indicateurs radio-actifs pour la détermination du débit de charriage des lits sableux.* Commissariat à l'énergie atomique (France). (CEA-RO3431).

Ziegler, C.A., *et al.* 1967. Radioisotope gauge for monitoring suspended sediment in rivers and streams. *Ijari,* vol. 18.

DISCUSSION

Grindley: The impression could arise that in the United Kingdom we can use as much radioactivity as we want to. On the contrary, it is very very difficult to get permission to put radioactive material into any of our rivers. On one occasion about 1956, 29 curies were put into the Thames esturary at a site where there were no fish within some 40 km. There is no water supply being taken from the site and no harm could possibly be done. The scandium was incorporated into glass particles, which were around the same size as the silt of the Thames so that this material was mostly near the bed of the estuary. However, in our Hydraulics Research Station at Wallingford, we have largely changed over to using fluorescent materials for tracing tests largely because of this difficulty of getting permission to use radioactive material in British rivers.

IX Special ships
Embarcations spéciales

Special hydrometric ships for the inland waterways of the Federal Republic of Germany

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Abstract. In recent years, a number of special ships have been built for hydrometric work on the inland waterways of the Federal Republic of Germany, in particular for the measurement of current speed, discharge, and bedload and suspended sediment transport. Water depth below the ship or above an area is sounded and graphically recorded with the aid of echo sounders, radiologs, depth recorders and bank distance sounders (horizontal sounders). Cross-sections are either reproduced immediately to scale, or recorded on punched tape for evaluation by the computer and printing to a desired scale by the plotter.

This paper describes the shape, dimensions, propulsion and draught of the special ships, and explains the measuring functions and procedures.

BATEAUX HYDROMETRIQUES SPECIAUX UTILISES PAR LES SERVICES DES EAUX INTERIEURES DE LA REPUBLIQUE FEDERALE D'ALLEMAGNE

Résumé. Au cours des dernières années, on a construit une série de bateaux spéciaux destinés aux travaux hydrométriques effectués sur les voies fluviales de la République fédérale d'Allemagne. Ces bateaux sont utilisés pour mesurer la vitesse et le débit des rivières ainsi que le charriage de fond et les sédiments en suspension. On détermine par sondage la profondeur de l'eau le long du trajet du bateau ou au-dessus d'une partie du lit fluvial au moyen d'une échographe, d'une radio-sonde, d'un enregistreur de profondeur et d'une sonde à mesurer la distance de la rive (sonde horizontale). Des sections transversales sont reproduites immédiatement à l'échelle voulue ou enregistrées sur bandes perforées pour traitement par compteur électronique et impression par des traceurs à l'échelle désirée.

Les auteurs donnent une description de la forme, des dimensions, de la force motrice et du tirant d'eau des bateaux spéciaux et fournissent des explications sur leur utilisation et leur fonctionnement.

EMBARCACIONES HIDROMÉTRICAS ESPECIALES EN LOS CURSOS DE AGUA CONTINENTALES DE LA REPÚBLICA FEDERAL DE ALEMANIA

Resumen. Recientemente se han construido cierto número de embarcaciones especiales para trabajos hidrométricos en los cursos de agua continentales de la República Federal de Alemania, en particular para la medición de caudal, caudal salido, arrastre de fondo y transporte de sedimentos en suspensión. La profundidad del agua por encima de la línea de rumbo de la embarcación o por encima de una zona de fondo se mide con la sonda y se registra gráficamente con ayuda de ecosondas, registradores de profundidad y sondadores de distancia a las orillas (sondadores horizontales). Con estos datos se reproducen a escala secciones trasversales o se registran sobre cinta perforada para evaluación por el ordenador e impresión, a la escala deseada.

La comunicación describe la forma, dimensión, utilización y proyecto de las embarcaciones especiales y explica los procesos de medición y metodología.

СПЕЦИАЛЬНЫЕ ГИДРОМЕТРИЧЕСКИЕ СУДА НА ВНУТРЕННИХ ВОДНЫХ ПУТЯХ ФРГ

Резюме. В последние годи изготовлен ряд специальных судов для гидрометрических работ на внутренних водных путях Западной Германии, в

частности, для измерения течений, расходов, транспорта донных и взвешенных наносов. Глубина воды по линии хода судна измеряется и графически записывается с помощью эхолотов, радиолагов, самописцев глубины и измерителей горизонтальных расстояний. Поперечные сечения либо немедленно воспроизводятся в масштабе, или записываются на перфоленту для последующей обработки на счетной машине и печати в требуемом масштабе специальным прибором (типа планиграфа).

В докладе описывается форма, размер, оборудование специальных судов и поясняются измерительные фунции и методика. Некоторые из этих судов встанут на якорь около здания Конгресса и могут быть осмотрены во время Симпозиума.

The Waterways Administration, which is subordinate to the Federal Ministry of Traffic, is required to improve and maintain the waterways of the Federal Republic of Germany and to ensure the safety and ease of movement of shipping. This requires numerous hydrometric measurements and investigations.

In addition to maintaining a comprehensive network of recording gauges, punched-tape gauges and tele-transmission gauges, the water levels of which are transmitted daily over the radio network, the Administration measures the discharge at every staff gauge. This, in turn, requires point or integrated measurements for the determination of the stage/discharge relationship which fluctuates as a result of erosion or aggregation.

For the benefit of shipping and river maintenance, flow velocity and flow direction are measured at various water stages, and the velocity distribution in the cross-section is determined. The surface velocity in the navigation channel is of special interest to shipping activities.

The transport of solid matter which may affect river bottom and bank zones is measured with the aid of bedload and suspended sediment samplers.

River bottom contours and their changes are determined through echo sounding. With the help of sounding leads and radiologs, river bottom profiles are established during longitudinal and transverse travel; these are either plotted directly or first recorded on punched tape and evaluated later by computer. A pair of transducers is placed at the centre of the ship's bottom, one emitting ultrasonic impulses and the other receiving the echo reflected from the bottom of the river.

Other special ships feature a number of transducers along their longitudinal axis, or two transversely arranged jibs carrying the transducers. They may, therefore, be employed for both the determination of cross profiles and area sounding.

Horizontal echo ranging equipment, the radio beams of which scan the distance between ship and banks at any required water depth, are used to measure the width of the watercourse or of the navigation channel. The equipment is also used for the location of shipping obstacles in the navigation channel and in harbours, and the checking of the statutory water depth.

DESIGN AND DRIVE OF THE SPECIAL SHIPS

Standardization of the special ships is impossible for the following two reasons: (a) the navigable waterways differ considerably in depth, flow and weather conditions; and (b) the variety of hydrometric measurements requires that the ship's form be adapted to the specific task. These reasons have led to the development and use of diversified types of ship and propulsion.

The conventional form of ship must be enlarged where measurements call for a stable and undisturbed equilibrium. If measurements are to be carried out close to the bank, the ship's draught must be small. Stability can also be obtained by using vessels in the form of pontoons or rafts, but this means reduced travelling speed. Vessels of the catamaran type possess the maximum stability in water.

Propulsion is by means of one or two variable-pitch or invariable-pitch propellers, swivelling propellers or propellers with vertical axis and adjustable blades (Voith-Schneider). Both types permit easy manœuvring within the smallest area. The ship's drive and the measuring instruments must be so arranged that the wake will not influence the measurements.

SPECIAL SHIPS WITH SEVERAL MEASURING FUNCTIONS

We shall first describe some ships which are fitted with a versatile range of equipment allowing them to be used for a variety of tasks.

Hydrometric ship "Herstelle" (catamaran)

Dimensions:

2 pontoons	length	20.95 m
	width	1.10 m
	height	1.05 m
	clear width between pontoons	3.00 m
	draught	0.70 m
Platform	length	19.00 m
	width	6.20 m
2 jibs	length	16.91 m
Propulsion:	1 swivelling propeller	
Engine rating:	120 h.p.	

The propulsion is located astern, between the two pontoons. The ship has two instrument cabins, one of them serving also as the pilot bridge. Two towline winches are mounted at the forecastle for swinging out the jibs, each of the latter being supported by two pontoons. With swing-out jibs, the total measuring width over which 40 echo transducers are distributed is 40 m, running trasversely to the ship's length.

The pilot room accommodates 1 depth map plotter, 1 cross-section recorder, 1 recorder for two horizontal echo ranging units, and 1 tachometer indicating the ship's travelling speed relative to the river bottom (all four instruments were supplied by the firm Atlas-Elektronik, Bremen). With these instruments it is possible to survey the waterway bottom quickly and accurately, to detect bars, and to control the work of dredgers.

A discharge measuring winch with counter has been installed in the forward cabin. The current meter is suspended from the measuring table and lowered between the two pontoons through a slot in the centre of the forecastle.

This ship is particularly well suited for narrow waterways. It has a normal speed of 8 km/hr and a speed of 5 km/hr when measuring. The ship is employed on the River Weser.

Hydrometric ship "Niederrhein"

(The ship is shown in Figure 1).

Dimensions:

length	27.80 m
width	6.71 m
draught	0.65 m

Propulsion: 2 variable-pitch propellers

Engine rating: 2 x 120 h.p.

Distance between propellers: 3.20 m.

The ship is held exactly in position by one head anchor and two side chains running to both banks. The corresponding winches are driven electrically, operation being controlled from the navigation cabin.

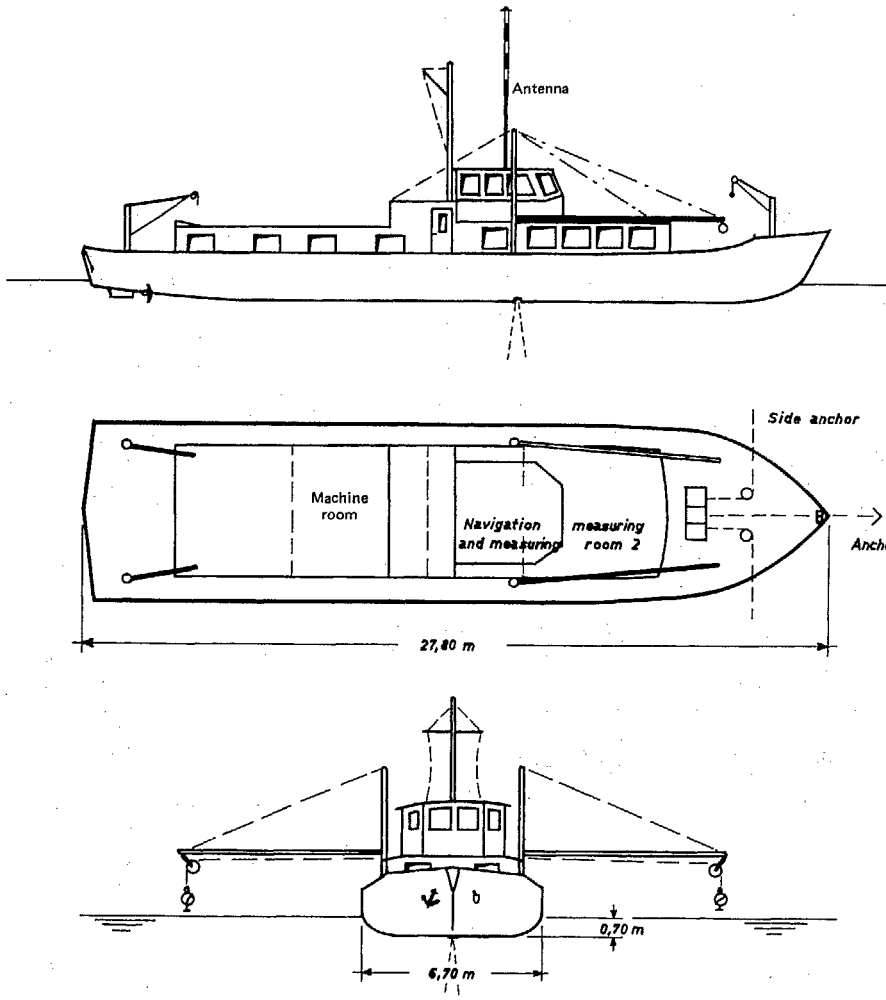


Figure 1 – *Measuring ship "Niederrhein"*

One pair of echo transducers is installed in the centre section of the ship's bottom. Installed in the upper pilot and measuring cabin is an Atlas surveying system (SUSY) consisting of a cross-section recorder, a digital indicator and a punched-tape recorder for the evaluation of the distance and depth values by computer. Cross-sections can be printed out by plotters, true to scale, together with the values for water stage and slope of the river banks. An arrangement for imprinting isobaths on river maps is still in the development stage.

A second cabin accommodates an electrically operated discharge measuring set-up comprising two automatically controlled winches from which the measuring cables run over two laterally arranged, swivelling telescopic jibs. In conjunction with two flow meters, they are used for carrying out point or intergrated measurements in

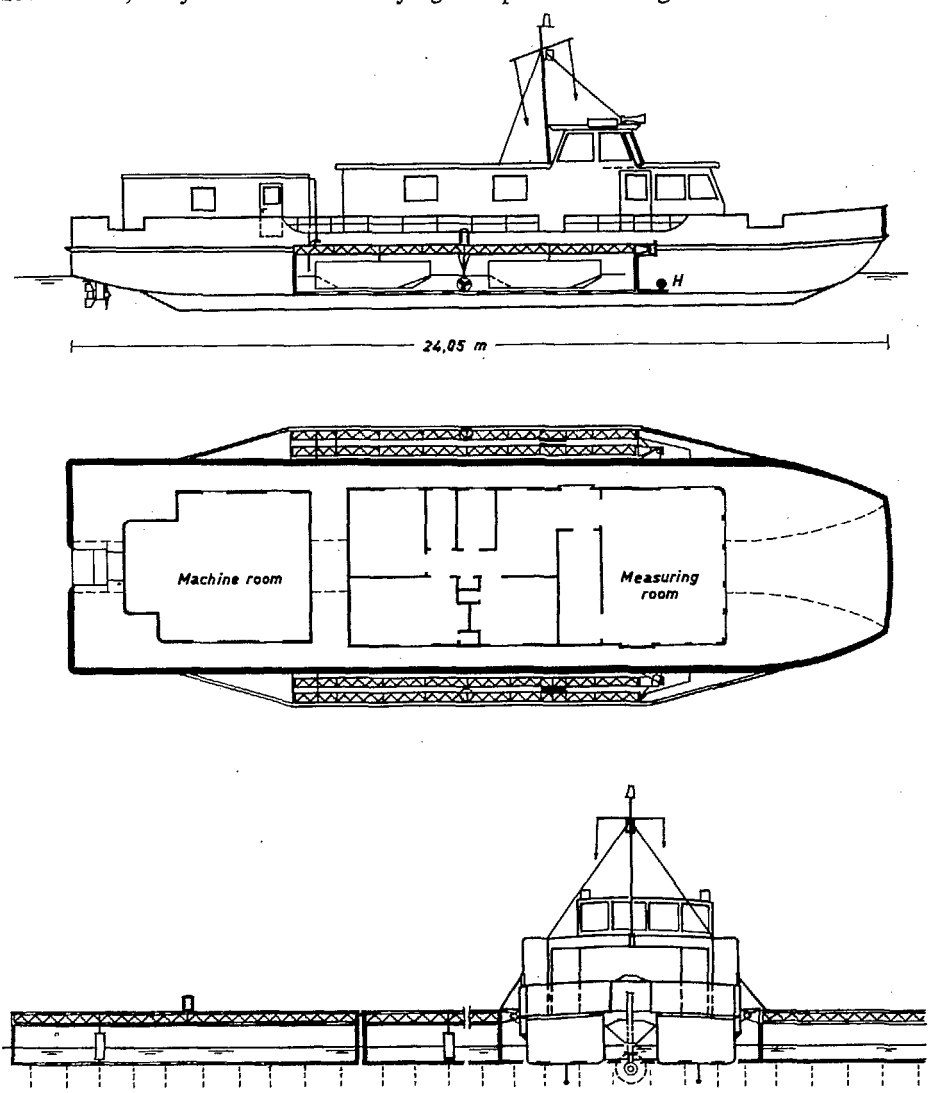


Figure 2 – *Measuring ship "Karl Hayssen"*

extended lateral positions of 15 m or 20 m. The integrated measurement can be obtained either during lowering or raising the current meter, so that a double measurement is possible within a very short time.

Two electrically operated jibs are provided on the quarter deck for lowering sediment and bedload measuring instruments into the water. For solid sediment and discharge measurements, the ship's position relative to the banks is fixed with the aid of the range finder section of the echo sounder.

The ship can also be used for carrying a levelling staff which is sighted from the banks at intervals of, for example, 500 m, for water level measurements. It has a speed of 18.7 km/hr and is employed on the Lower Rhine.

Hydrometric ship "Walter Tuerk"

This ship has the same dimensions as the measuring ship *Niederrhein,* and is similarly equipped with measuring instruments. It is employed on the Middle and Upper Rhine.

We shall now describe several ships which are used primarily for measuring and controlling the condition of the bed of the waterway and of the navigation channel.

Hydrometric ship "Mimir"

Dimensions:

length	20.00 m
width	8.00 m
maximum draught	0.90 m
2 jibs, length	16.50 m

Propulsion: 2 Voith-Schneider propellers astern

Engine rating: 2 × 75 h.p.

Distance between propellers: 3.15 m

A double winch is mounted on the forecastle for hauling the two jibs which are supported by four floats each. With the jibs extended, the measuring width is 41; Over this width, a total of 41 echo transducers is distributed; the latter are arranged underneath the jibs and the bottom of the ship. A Doppler-log indicates the ship's course and travelling speed relative to the river bottom.

The instrument cabin accommodates an area echograph (from Dr. Farenholz, Kiel), which can also be used for recording cross-sections. The ship is particularly well suited for the control of waterways of limited width. Speed is 15.0 km/hr; with the jibs extended, speed decreases to 7.0 km/hr. The ship is employed on the River Main.

Hydrometric ship "Mosel"

Dimensions:

length	30.12 m
width	7.16 m
draught	1.08 m
2 jibs, length	16.50 m

Propulsion: 2 Voith-Schneider propellers astern

Engine rating: 2 × 150 h.p.

The hollow jibs are fitted with a float at the outer end. They are arranged below the water surface and are operated hydraulically. The measuring width with jibs extended is 40 m. Underneath the jibs and their extension below the ship's bottom, 54 echo transducers are arranged at intervals of 1.00 m and, in some cases, of 0.50 m. The navigation and instrument cabin accommodates a bottom map recorder (Atlas, Bremen) which plots a correctly scaled relief map in different shades of grey while the ship is moving, also recording cross profiles at regular intervals. In order to protect the tips of the jibs from contact with the ground, the water depths at these tips are measured by small echo sounders, and indicated in the navigation cabin.

The ship has been designed for waterways of limited width. Its speed is 18.0 km/hr, which is reduced to 6 km/hr during measurement. The ship is employed on the River Moselle.

Hydrometric ship "Ruhrort" (pontoon type of trapezoidal cross-section)

Dimensions:

length	34.20 m
width	6.50 m
draught	0.80 m

Propulsion: 2 Voith-Schneider propellers

Engine rating: 2 × 85 h.p.

One of the propellers is arranged in the forecastle, the other astern so that the ship can turn in its own length or travel sideways. Below the ship's bottom, 41 echo transducers are arranged in a longitudinal pattern.

The navigation and measuring cabin accommodates a bottom map recorder (Atlas, Bremen). When the ship travels sideways, the instrument plots, in five different shades of grey on two depth map tapes, the high-bed areas as well as the depths over them. The depths are referred to an adjustable standard depth, and the measurements are made over a width fo 20 m. Scaled cross-sections are recorded with the aid of an echolog (Atlas).

The speed of the vessel is 13.0 km/hr normally and 7.2 km/hr when traversing. The ship is used for monitoring the bed of the Lower Rhine.

Hydrometric ship "SF III"

Dimensions:

length	35.00 m
width	11.12 m
draught	1.10 m

Propulsion: 4 vertical propellers

Engine rating: 4 × 85 h.p.

Originally built as a ferry, this ship is quite wide. It is very stable and, therefore, particularly well suited for depth measurements. Sixty-one transducers are arranged on one side in a rake pattern. The instruments aboard are a bottom map recorder with cross-section recorder, and an echolog which provides directly scaled pictures. The four laterally arranged vertical propellers allow smooth sideways travel during measuring. The ship is used on the Rhine.

Hydrometric ship "Karl Hayssen" (catamaran)

(This measuring ship (Fig. 2) was built especially for the maintenance of the "Nord-Ostsee-Kanal" (the so-called Kiel canal between the Baltic Sea and the North Sea) which can be used by ocean-going ships of up to 9.5 m draught. The canal has a nominal depth of 11.00 m and a very narrow channel. The heavy increase in traffic (1969: 78.6 million gross registered tons), due primarily to the passage of large-draught ships, leads to bank erosion and wash-out so that landslips of considerable size can occur. In such cases, the damage must be immediately determined with the aid of the echo sounder, and the canal profile must be restored by dredging as quickly as possible).

Dimensions:

length	24.05 m
width	8.30 m
draught	0.60 m
2 floats, width	2.25 m
clear width between floats	1.50 m
2 jibs, length, half extended	11.00 m
length, fully extended	22.00 m

Propulsion: 1 vertical propeller astern

Engine rating: 200 h.p.

A bow jet rudder ensures improved manœuvrability during landing and departing operations. Deckhouse and machinery are installed vibration-free so as to achieve the necessary noise-dampening effect.

Jib extension is effected diesel-electrically. The jibs can be folded once so that the following measuring widths are available:

both jibs fully extended	50 m
both jibs half extended	30 m
one jib fully extended	19 m.

Forty-eight echo transducers are arranged underneath the jibs and the ship. The instrument cabin accommodates the following equipment:

1 two-sided echo ranging gear for measuring the distance between the banks and the ship;
1 area-recording echo sounder;
1 profile-recording echo sounder;
1 navigation sounder.

The echo ranging near records on a 860 mm wide paper tape the distance between the tips of the jibs, or the sides of the ship, and the banks at a scale of 500:1.

The area-recording echo sounder plots sounded depth lines and variable nominal depths so that a depth map can be drawn instantaneously. The profile-recording echo-sounder plots cross-profiles, on a 100 : 1 scale, within the set measuring width. By depressing a pushbutton, all instruments (from Dr. Fahrenthoz, Kiel) make a time mark on all records to facilitate identification. For measuring sound velocity and regulating the measuring instruments, a 10 m long calibration length is provided between the catamaran's two floats.

The measuring ship has a speed of 15.5 km/hr, which is reduced to 7.4 km/hr for measuring operations.

Advantages include: excellent heeling stability; a stable course even at lowspeeds; very large lateral surface; good manœuvrability; small draught; and, good dampening of machine noise.

The ship was commissioned in 1968 and has worked completely satisfactorily ever since.

Hydrometric boat "Vollerwick"

Dimensions:

length	11.80 m
width	3.20 m
draught	0.80 m

Propulsion: 1 propeller

Engine rating: 75 h.p.

The navigation and measuring cabin accommodates a profile-recording echo sounder for depth measurement, and a HiFix instrument for position fixing. The boat has a speed of 15.5 km/hr and is particularly well suited for measurements in the shallow waters of the shoals. It is employed primarily on the River Eider.

Hydrometric boat "Echo II"

Dimensions:

length	10.00 m
width	2.70 m
draught	0.40 m

Propulsion: 1 propeller

Engine rating: 40 h.p.

The measuring boat is equipped with a profile-recording echo sounder, a horizontal echo ranging gear and a special winch. It travels backwards across the watercourse, pulling a measuring rope fastened to the bank. After the opposite bank has been reached, the rope is pulled in again so that the boat hauls itself back into the starting position. The rope winch also drives the recording paper tape so that the cross profiles obtained are correctly scaled.

The boat has a speed of 12 km/hr and is employed in coastal regions.

X Remote sensing
Détection à distance

Temperature measurements of water surfaces using infra-red radiation thermometers

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Federal Republic of Germany

Abstract. Most of the infra-red radiation thermometers used up to now in meteorology and hydrology have a spectral bandpass from 8 μm to 13 μm or 14 μm. It is demonstrated that a narrower bandpass, e.g. from 9.5 μm to 11.5 μm is more convenient for the temperature measurements of water surfaces. A radiometer with this narrower bandpass was used in an aircraft flying over water. The results of these measurements are discussed.

MESURE DE LA TEMPERATURE DES SUPERFICIES D'EAU AU MOYEN DE THERMOMETRES A RAYONNEMENT INFRAROUGE

Résumé. La sensibilité spectrale de la plupart des thermomètres à rayonnement infrarouge utilisés jusqu'à présent pour les mesures de température en surface dans le domaine de la météorologie et de l'hydrologie atteint de 8 μm à 13 μm ou même 14 μm de longueur d'onde. On démontre que, pour les mesures de la température des eaux de surface, un intervalle plus étroit, par exemple de 9,5 μm à 11,5 μm, est plus favorable. Un radiomètre utilisant cet intervalle spectral plus étroit a été utilisé par des avions au-dessus de l'eau. Les résultats obtenus font l'objet d'un examen critique.

MEDICIÓN DE LA TEMPERATURA DE LAS SUPERFICIES DE AGUA CON TERMÓMETROS DE RAYOS INFRARROJOS

Resumen. La mayoría de los termómetros de rayos infrarrojos usados hasta la fecha en meteorología e hidrología tienen un paso de banda espectral de 8 μm a 13 μm o 14 μm. Se ha demostrado que un paso de banda más estrecho, p. ej. de 9,5 μm a 11,5 μm, es más conveniente para las mediciones de temperatura en las superficies de agua. Se utilizó un radiómetro con este paso de banda más estrecho midiendo desde un avión y en esta comunicación se discuten los resultados de las mediciones efectuadas.

ИЗМЕРЕНИЕ ТЕМПЕРАТУРЫ ВОДНОЙ ПОВЕРХНОСТИ ПУТЕМ ИСПОЛЬЗОВАНИЯ ИНФРАКРАСНЫХ РАДИОАКТІВНЫХ ТЕРМОМЕТРОВ

Резюме. Большиство из инфракрасных радиоактивных термометров, используемых по настоящее время в метеорологии и гидрологии, реагируют на прокождение лучей спектра от 8 μм до 13 μм или 14 μм. Это показывает то, что более узкая полоса прохождения, например, от 9,5 μм до 11,5 μм является более подходящей для температурных измерений водных поверхностей. Радиометр с этой более узкой полосой прохождения был использован в воздушных полетах над водой. Результаты этих измерений излагаются в докладе.

INTRODUCTION

The radiation method for measurement of the surface temperature of water has been developed over the last few years. Its airborne use offers the possibility of measuring large areas in a relatively short period of time. Such surveys are nowadays routine (Grossman *et al.*, 1969; Lorenz, 1969b).

The method exploits the relationship between the temperature of a surface (less than 0.1 mm layer) and its emitted long wave radiation. Most of the infra-red radiation thermometers (IRT) employed for this purpose have a spectral bandpass from 8 μm to 14 μm, which covers one of the "window regions" of the atmosphere. This ensures good visibility and accordingly small interference by absorption and emission by the air layer between the target and the IRT. Part b of Figure 1 (filter B) shows the spectral sensitivity of a radiometer of this type and, below (c), the spectral transmission of a 300 m layer of the atmosphere after measurements from Yates and Taylor (cited from Bolle, 1964). In the range 8 to 14 μm, the IRT exceeds the limits of the atmospheric window. Thus absorption and emission by the atmosphere are not to be ignored, at least at distances of 300 m or more. Correction methods had to be derived (e.g. Saunders, 1967; Lorenz, 1968, 1969a; where Saunders' method includes the reflection correction mentioned below). They yield, in the case of water surfaces, corrections of less than + 1° at a flight level of 1000 ft (~ 300 m) and between − 1° and + 2° at a flight level of 3000 ft (~ 900 m).

A restriction of the spectral bandpass so that it is totally within the limitations of the "window" minimizes this correction. Until recently this was impracticable because of the low fluxes of energy involved. Nowadays advances in technology have eliminated this problem.

There is another reason for the limitation of the spectral bandpass of the IRT. Water is not really black in this spectral region, but has a small reflectivity as shown in Figure 2 (after Kislovskii, 1959). It reflects a small amount of the infra-red sun and sky radiation, which influences the surface temperature measurements. The steep ascent of the reflectivity beyond 11 μm (Fig. 2) suggests that the spectral bandpass should not be extended to this region (Lorenz, 1966, 1968). Therefore an IRT with a 9.5 μm to 11.5 μm bandpass as shown in a part of Figure 1 (filter A), seems to be particularly appropriate.

The object of the investigation was a comparison of the two spectral bandpasses, 8 to 14 μm and 9.5 to 11.5 μm. For this, an IRT from Barnes type PRT 6 was used, which is specially fitted with a filter wheel for different infra-red filters. In addition, theoretical calculations were made and compared with the results of the measurements.

INFLUENCE OF REFLECTIVITY

Since a water surface produces specular reflections and IRT measurements of water normally are made vertically from above, solar radiation is reflected from smooth water surfaces into the IRT only when the position of the sun is close to the zenith. From a rough sea and at different positions of the sun errors appear in exceptional cases only (Saunders and Wilkins, 1966; Saunders, 1967). Therefore only the reflected atmospheric sky radiation has to be taken into account. Its influence is greatest when the sky is cloudless because the sky radiation is very low in the window region of the atmosphere. On this account a correction diagram of the influence of the

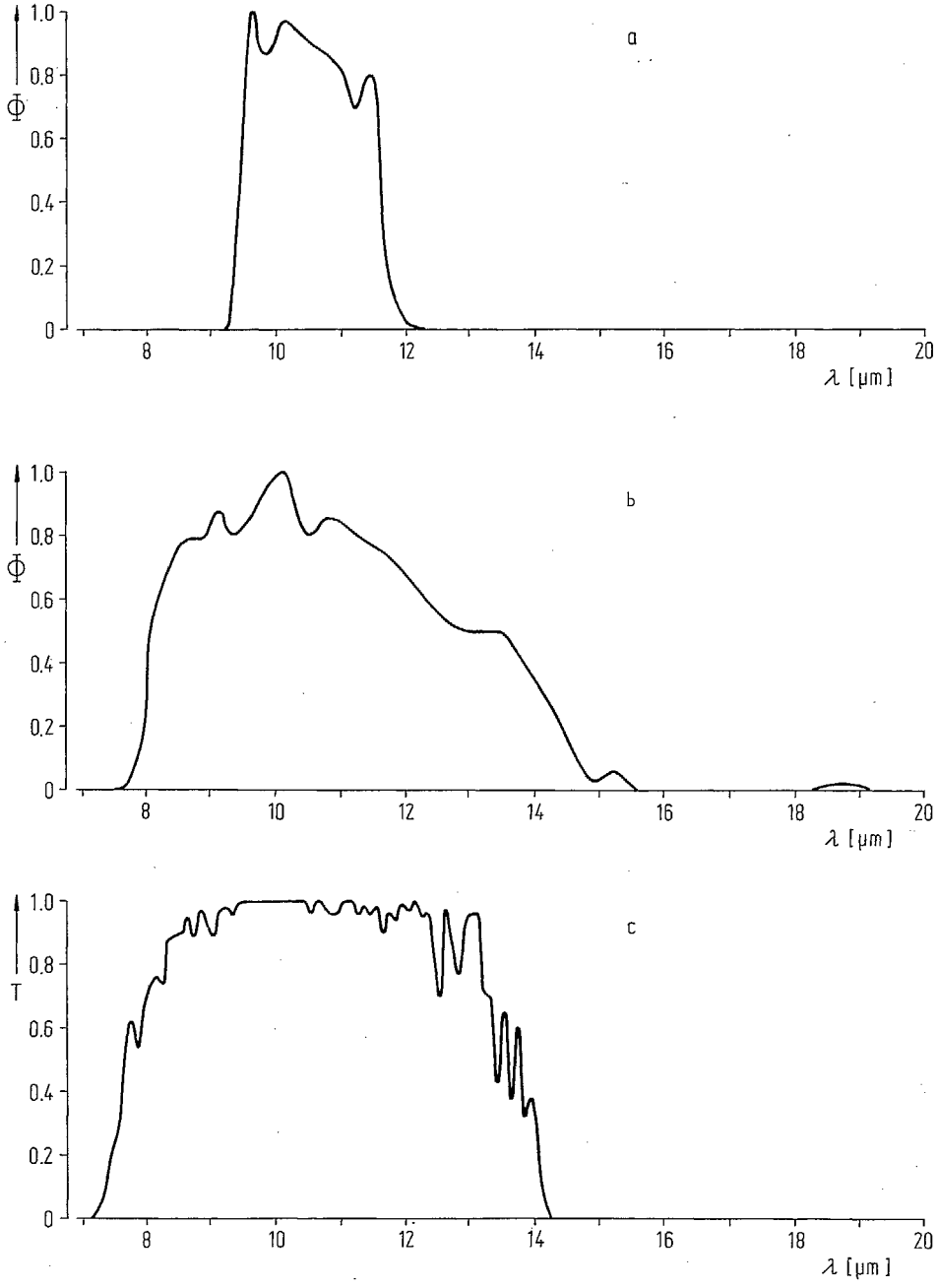


Figure 1 – *Spectral sensibility* Φ *of the PRT 6 with filters A and B (sections a and b) and spectral transmission T of the atmosphere for the same spectral region (section c) for an optical path length of 300 m, 26 °C air temperature and 0.57 cm ppw (after measurements of Yates and Taylor, see Bolte, 1964)*

reflected sky radiation of the cloudless sky to the 9.5 to 11.5 μ m spectral region was computed. This was done in the same way as for the 8 to 14 μm region (Lorenz, 1966, 1968).

The difference between the true surface temperature θ and the so-called radiation temperature θ_S (this is the surface temperature as measured with an IRT without correction of the reflection influence) may be calculated by the following equation:

$$\int_{\lambda_1}^{\lambda_2} \Phi_\lambda \,.\, E_\lambda(\theta)\,\mathrm{d}\lambda - \int_{\lambda_1}^{\lambda_2} \Phi_\lambda \,.\, E_\lambda(\theta_s)\,\mathrm{d}\lambda$$
$$= \int_{\lambda_1}^{\lambda_2} \Phi_\lambda\,(1-\epsilon_\lambda)\,E_\lambda(\theta)\,\mathrm{d}\lambda - \int_{\lambda_1}^{\lambda_2} \Phi_\lambda\,(1-\epsilon_\lambda)\,.\,G_\lambda\,\mathrm{d}\lambda\,. \quad (1)$$

Φ_λ is the spectral sensitivity of the IRT with the wave-length limits λ_1 and λ_2, $E_\lambda(\theta)$ the spectral radiative energy of a black body of temperature θ, ϵ_λ the spectral emissivity of water, and G_λ the spectral atmospheric sky radiation. $E_\lambda(\theta)$ and G_λ refer to the same solid angle.

The atmospheric sky radiation G_λ of the cloudless sky for the calculation of the second integral on the right-hand side of equation 1 was taken from a number of spectral measurements of the zenith radiation (Lorenz, 1966). The numerical values of these integrals were plotted versus the air temperature at the time the spectra were recorded (Fig. 3). The relationship between reflected sky radiation and air temperature was satisfactory, so that it was possible to compute a correction diagram for the reflectivity influence in which the differences between true surface temperature and radiation temperature $\theta - \theta_S$ were plotted versus radiation temperature θ_S and air temperature θ_L (Fig. 4). The values are between + 0.3° and + 0.5° for differences between water temperature and air temperature of not more than 10° which rarely may be exceeded (not screened part of Fig. 4).

Figure 5 shows a comparison with the 8 to 14 μm spectral bandpass. The differences between true surface temperature and radiation temperature $\theta - \theta_S$ are plotted versus surface temperature for the case of surface temperature equal to air temperature. The curve of filter B (8 to 14 μm) was drawn after Lorenz (1968). The reduction of the influence of the reflectivity when changing from filter B to filter A shows clearly. The effect is large at low temperatures. Between 0°C and 5°C the differences $\theta - \theta_S$ obtained using filter B are 1.5 times those obtained using filter A. Up to 25°C this factor decreases to 1.2.

Figure 5 comprises also a curve for a monochromatic radiometer at 11 μm, the wave-length with the minimum of reflectivity of water and thus the minimum difference $\theta - \theta_S$. The curve of filter A comes close to that of the 11 μm radiometer. The 9.5 to 11.5 μm spectral region consequently is not far away from the ideal. This may also be shown in another way. The differences of the two integrals on the right-hand side of equation 1 are computed versus wave-length for a fictitious radiometer with a spectral bandpass from 7 to 15 μm. From this the percentage of 0.5 μm intervals of the total difference $\theta - \theta_S$ may be derived. The results are given in Figure 6 for 12°C surface temperature and a corresponding infra-red sky radiation. The steep ascent of the influence of reflectivity beyond 11.5 μm is clear.

These results refer to a smooth water surface. In the case of ocean water the reflectivity increases (Friedman, 1969), but this has little effect on the results of measurements. Even a rough water surface causes modifications. On the one hand the reflectivity increases; on the other hand the sky radiation reflected now originates

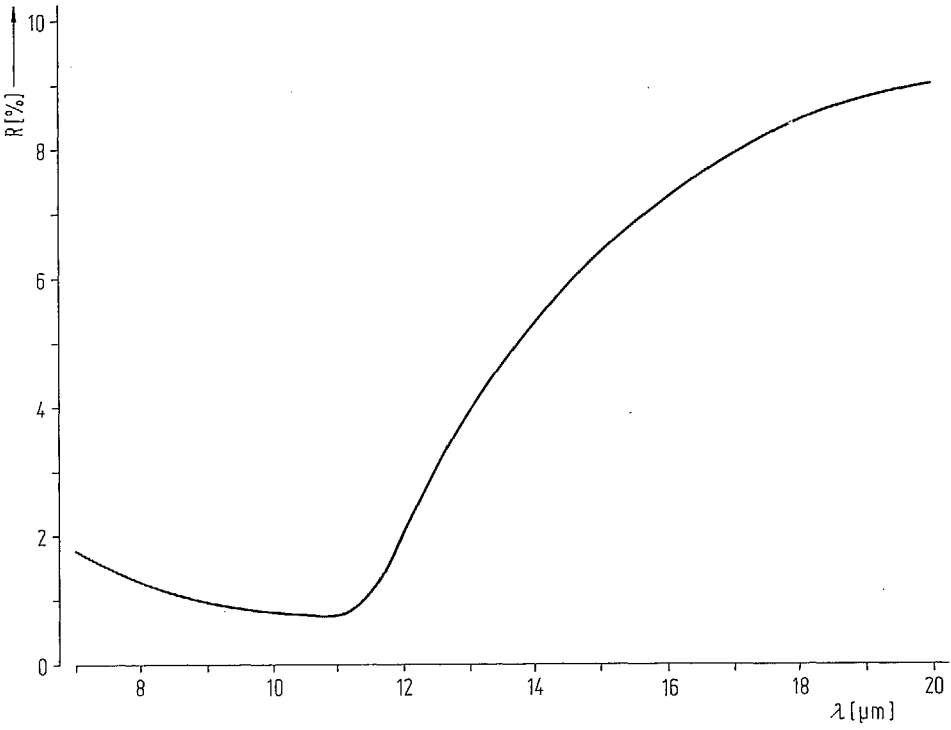


Figure 2 – *Spectral reflectivity of water (after Kislovskii, 1959).*

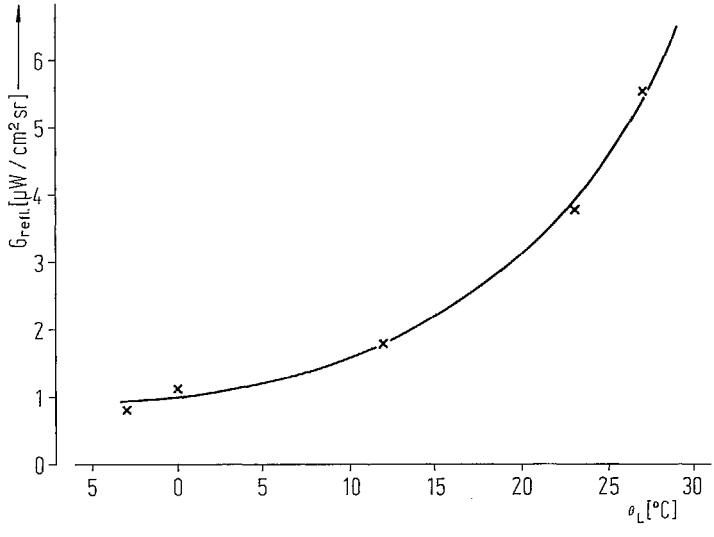


Figure 3 – *Reflected infra-red radiation of the clear sky effective to the PRT 6 with filter A (λ = 9.5 – 11.5 µm)*
$G_{refl.} = \int_{\mu_1}^{\lambda 2} \Phi_\lambda (1 - \epsilon_\lambda)\, G_\lambda \, d\lambda$ *versus air temperature* ϑ_L *for a smooth water surface*

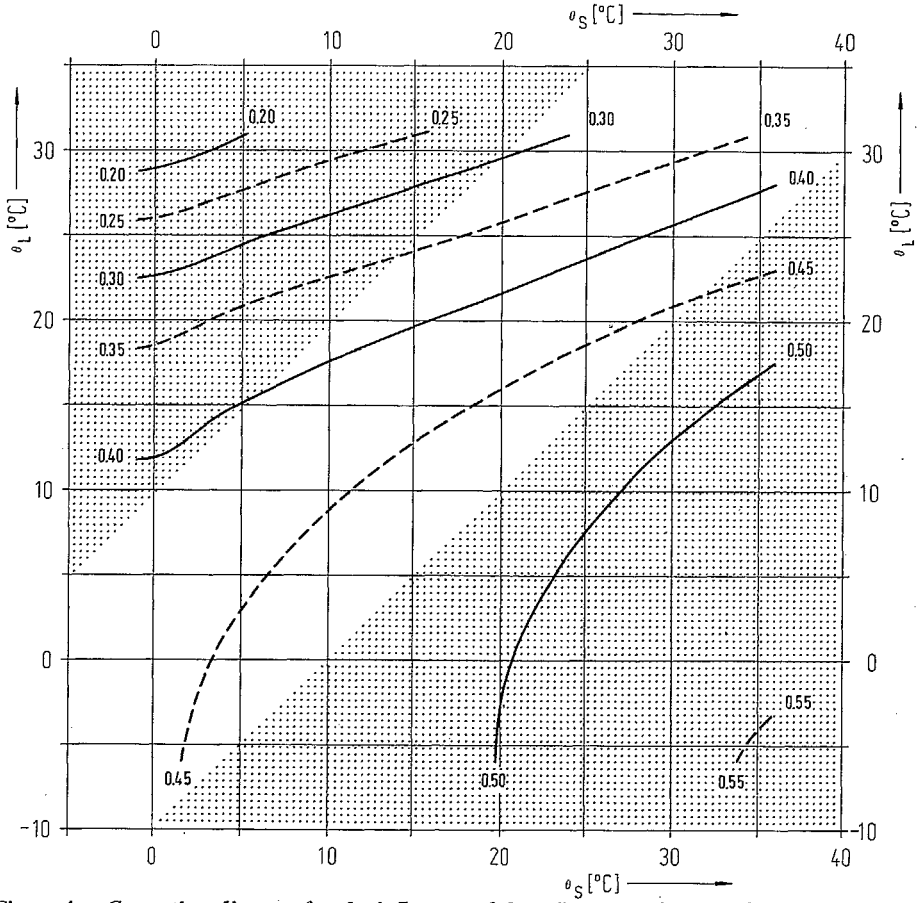


Figure 4 – *Correction diagram for the influence of the reflectivity of a smooth water surface on PRT 6 with filter A (9.5 to 11.5 µm) under clear sky conditions. Differences between true surface temperature and radiation temperature $\theta - \theta_S$ versus radiation temperature θ_S and air temperature θ_L*

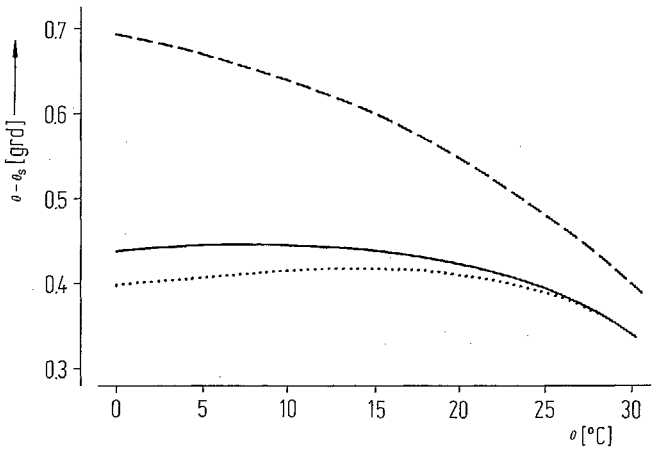


Figure 5 – *Difference between true surface temperature and radiation temperature $\theta - \theta_S$ versus surface temperature θ for a smooth water surface in case of surface temperature = air temperature.*

——— *PRT 6, Filter A,* ..... *PRT 6, Filter B (= PRT 4),* ..... *monochromatic radiometer for 11 µm wave-length*

Table 1. *Differences between true surface temperature θ and radiation temperature θ_S from measurements and from correction diagram. Measurements of stirred water at the Hohenpeissenberg. Sky cover* $<1/10$. $\theta_L \simeq$ *Air temperature.*

Date	Time CET	θ_L [°C]	θ [°C]	Filter	$\theta - \theta_S$ Measurement	$\theta - \theta_S$ Correction diagram
29 October 1969	1514	10.5	10.5	A	0.38	0.44
				B	0.58	0.63
31 October 1969	1455	7.0	4.8	A	0.33	0.44
				B	0.58	0.64

not from the "cold" zenith but from another "warmer" part of the sky. (The infra-red sky radiation increases in this spectral region from the zenith to the horizon.) Taken together the consequences to the differences between surface temperature and radiation temperature $\theta - \theta_S$ are negligibly small (Saunders and Wilkins, 1966). The correction diagram of Figure 4 is useful for these cases too.

The results of a comparison between measured differences $\theta - \theta_S$ and those derived from Figure 4 are shown in Table 1. (The lack of space does not allow a description of these measurements.) The values are in good agreement.

A cloud covered sky reduces the differences $\theta - \theta_S$. In exceptional cases, when the water temperature is lower than the temperature of the cloud layer, the corrections $\theta - \theta_S$ may even become negative. A useful estimation gives the following equation (Lorenz, 1968):

$$\theta - \theta_S \simeq (1 - \epsilon)\,(\theta_S - \theta_C) \qquad (2)$$

where $(1-\epsilon)$, the reflectivity of water, is, for the spectral region of filter A, 0.9×10^{-2}. θ_C is the temperature of the cloud layer for which an estimated value according to the cloud height is sufficient.

Difficulties result from changes between low clouds and a cloudless sky. An IRT record may simulate variations of the surface temperature caused by the variations of the (reflected) sky radiation. Saunders (1967) too pointed to this effect. Figure 7 shows an example for the influence of a single cloud on the airborne surface temperature record with a PRT 6. The spectral region of filter A with a simulated surface temperature variation of 0.45° under the cloud shows up to advantage against filter B which produces a 0.67° variation. This effect may be avoided by a synchronous record of the sky radiation during flight. Because of the small reflectivity of water, high accuracy of this is not essential.

ATMOSPHERIC ABSORPTION AND EMISSION

The influence of atmospheric absorption and emission was investigated experimentally. The altitude dependency of the surface temperature was investigated for both filter regions of the PRT 6 by airborne measurements over the Starnberger See, a lake about 25 km south-west of Munich. On a fixed flight path the levels were varied between 100 ft ($\simeq$ 30 m) and 10,000 ft (approx. 3,000 m) above the water surface. There were no difficulties in extrapolating the resulting curves of the radiation temperature θ_S versus altitude H to the radiation temperature at the level of the water $\theta_S(0)$. The differences between the radiation temperature at the altitudes of

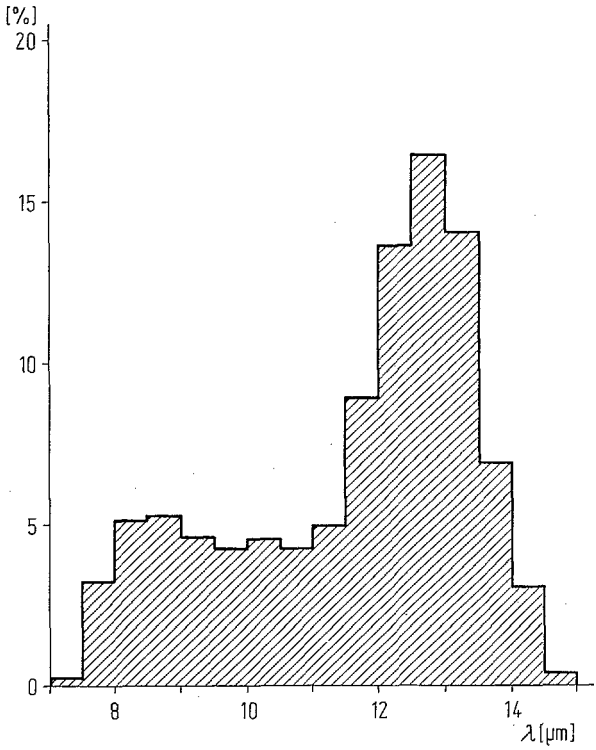


Figure 6 – *Percentage of 0.5 μm wave-length intervals to the difference between true surface temperature and radiation temperature* $\theta - \theta_S$ *for a radiometer with a spectral bandwidth from 7 to 15 μm. Surface temperature and air temperature 12° C, cloudless sky*

Table 2. *Differences between the radiation temperatures of the altitudes 0 and H* $\theta_S(0) - \theta_S(H)$ *in deg. from measurements and from correction diagram*

Date 1969	Time CET	Filter A		Filter B	
		Measurement	Correction diagram	Measurement	Correction diagram
H = 1 000 ft (≈ 300 m):					
5 Aug.	1 153	0.4	0.2	0.55	0.3
5 Aug.	1 500	0.1	0.0	0.25	0.0
7 Aug.	1 035	0.05	0.2	0.25	0.3
19 Sept.	1 303	0.3	0.5	0.55	0.8
25 Sept.	1 237	0.0	0.1	0.0	0.1
29 Sept.	1 041	0.3	0.3	0.5	0.5
H = 3 000 ft (≈ 900 m):					
5 Aug.	1 153	0.95	0.9	1.7	1.4
5 Aug.	1 500	0.7	0.8	1.05	1.3
7 Aug.	1 035	0.8	0.8	1.45	1.3

1,000, 3,000, 5,000 7,000 and 10,000 ft (≃ 300, 900, 1,500, 2,100 and 3,000 m) were determined. These values for filter B and filter A were plotted (Fig. 8). They are almost exactly on a straight line with a slope of 1.6 for all heights. This means that the differences $\theta_S(0) - \theta_s$ (H) of filter B are 1.6 times those of filter A. Thus the advantages of the limitation of the spectral bandpass to 9.5 to 11.5 μm are evident also with respect to the influence of the atmosphere.

Since the difference θ_s (0) – θ_s (H) of the two filter regions show linear dependency (Fig. 8) it is possible to use the correction diagrams for the 8 to 14 μm region (Lorenz, 1968, 1969a) for the 9.5 to 11.5 μm region after multiplication with $1/1.6 = 0.62$. Table 2 contains a comparison of measured differences θ_S (O) – θ_S (H) with the values from the correction diagram. The numbers of the column "correction diagram" of filter A are those of filter B multiplied by 0.62. The agreement of measured and computed values is very satisfactory, particularly when the accuracy of the measurements with the PRT 6, in this case about ± 0.2°, is considered.

EXAMPLES OF AIRBORNE MEASUREMENTS

Reservoir near Lechbruck (Upper Bavaria). The lake behind Lech Dam No. 3 is one of the numerous reservoirs of the river Lech which were constructed in recent decades. It is a little more than 2 km long and 830 m wide at the dam.

Airborne measurements of this reservoir were made along the 8 flight paths plotted in Figure 9 on 13 March 1970 between 1125 and 1139 h Central European Time (CET), i.e. within a quarter of an hour. The PRT 6 was used with filter A (9.5 to 11.5 μm). The flight level was 300 ft (≃ 90 m) above the water surface so that the target area was about 5 m × 5 m. Because of the low flight level there were no corrections for absorption and emission by the atmosphere. Since the sky was nearly cloudless the reflectivity corrections were taken from the diagram of Figure 4.

The results showed which parts of the reservoir were ice covered. Furthermore, it was possible to plot the isotherms of the water surface temperature (Fig. 9). The highest temperatures along the longitudinal axis of the lake are close to the former river bed of the Lech, near the left-hand shore. Because of the heavy turbulence of the water near the inlet of the electric plant, the surface temperature was almost the same as in the tributary of the Lech river. In other parts of the dam was an ice sheet, the break-up of which was caused by the flow of the Lech near the right-hand shore. In addition at one place a hole had been broken into the sheet and also small breaks (marked with x in the figure) were identified.

Bays of Kiel and Eckernrnförde (Baltic Sea). The measurements were made on 12 November 1969 between 1140 and 1220 h CET and worked out in the same manner as those described above. The flight level was 500 ft ≃ 150 m) and the target area accordingly about 7.5 m × 7.5 m. The isotherms (Fig. 10) were nearly parallel to the shores with surface temperatures increasing into the bays.

Bay of Heligoland (North Sea). This flight, made on 11 November 1969 between 1050 and 1410 h CET, took much longer than the two described above because of the enlargement of the measured area. Therefore the flight routes (Fig. 11) were defined with as many crossing points as possible in order to find the temporal changes of the surface temperature. On this day they were found to be negiligible.

The surface temperatures seem to be dependent on the depth of water (Fig. 11).

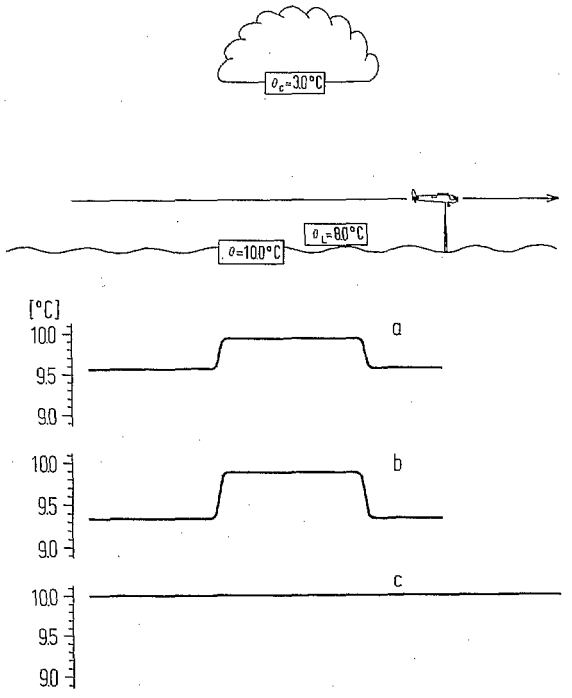


Figure 7 – *Example for simulated surface temperature variations caused by change from cloudless sky to low clouds* θ = *surface temperature of water,* θ_L = *air temperature,* θ_C = *cloud temperature. Records: a = PRT 6, filter A; b = PRT 6, filter B; c = true surface temperature*

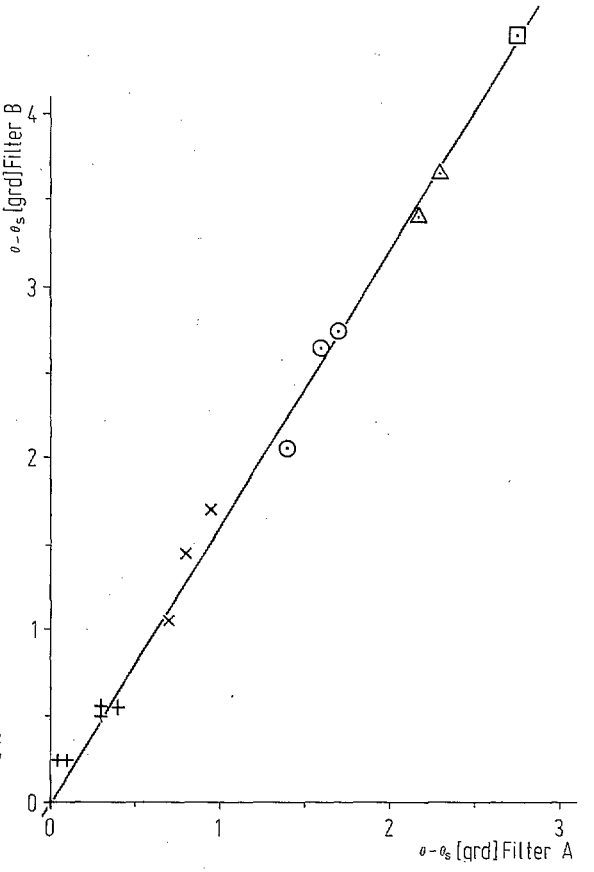


Figure 8 – *Differences between the radiation temperatures of the altitudes O and H* $\theta_S(O) - \theta_S(H)$ *for PRT 6, filter B versus Filter A.* + *H = 1,000 ft,* × *H = 3,000 ft,* ⊙ *H = 5,000 ft,* △ *H = 7,000 ft,* ⊡ *H = 1,000 ft*

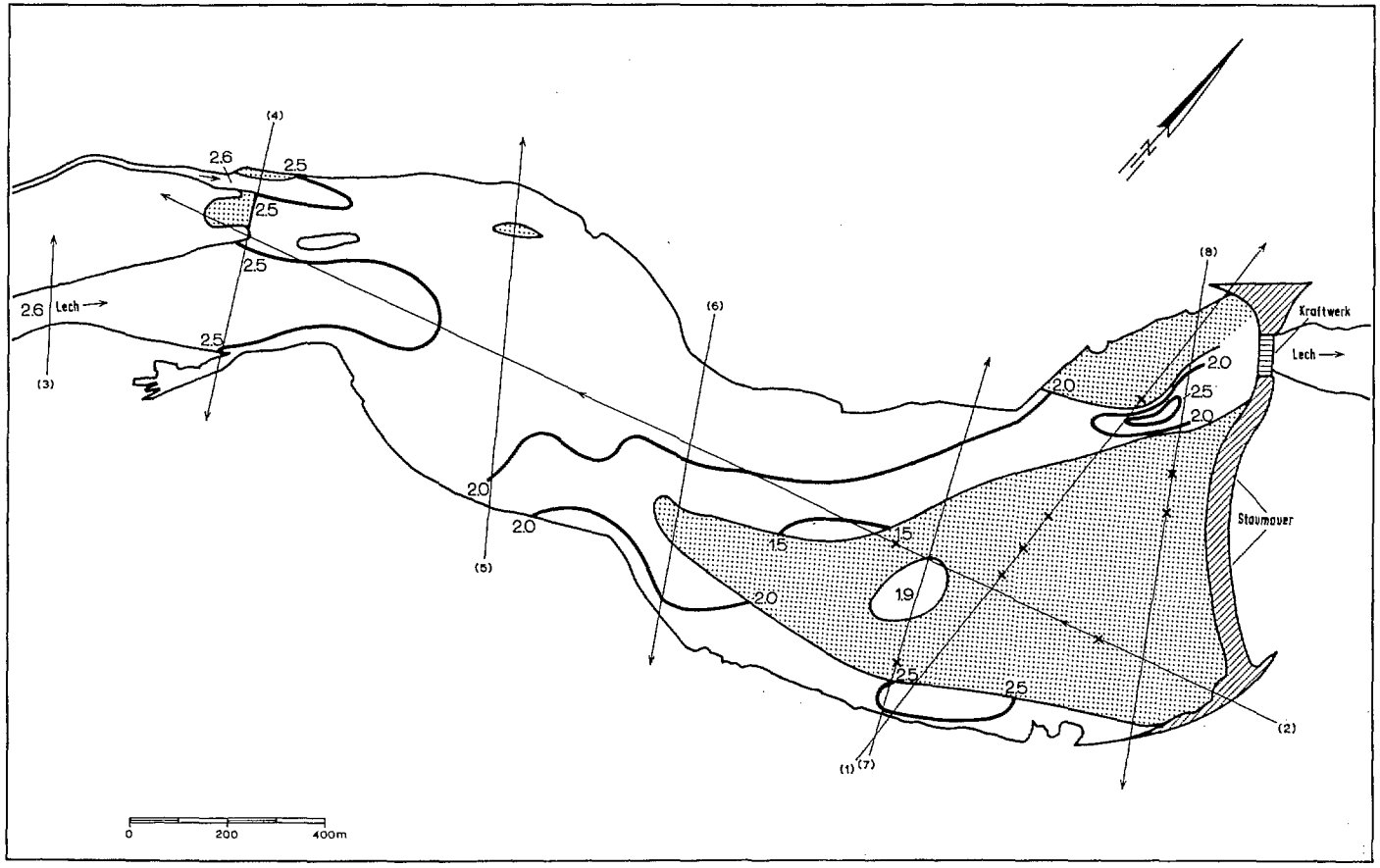


Figure 9 – *Ice and surface temperature distribution of the reservoir near Lechbruck 13 March 1970, 1125 to 1139 hr CET. Surface temperature in °C. Screened areas = ice cover, x = points with breaks of the ice sheet, (1) – (8) = flight paths*

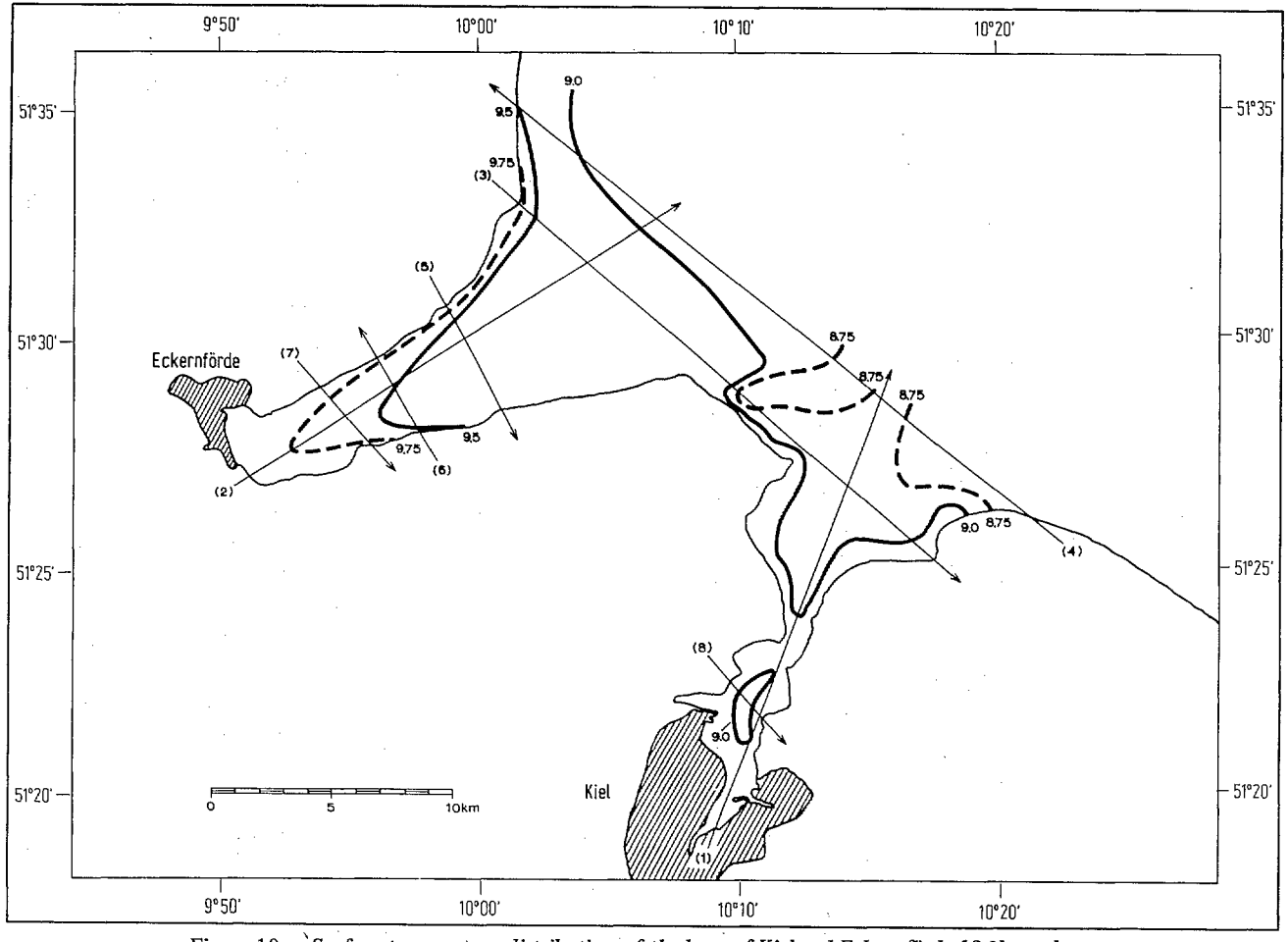


Figure 10 – *Surface temperature distribution of the bays of Kiel and Eckernförde 12 November 1969, 1140 to 1220 hr CET. Surface temperature in °C, (1) – (8) = flight paths*

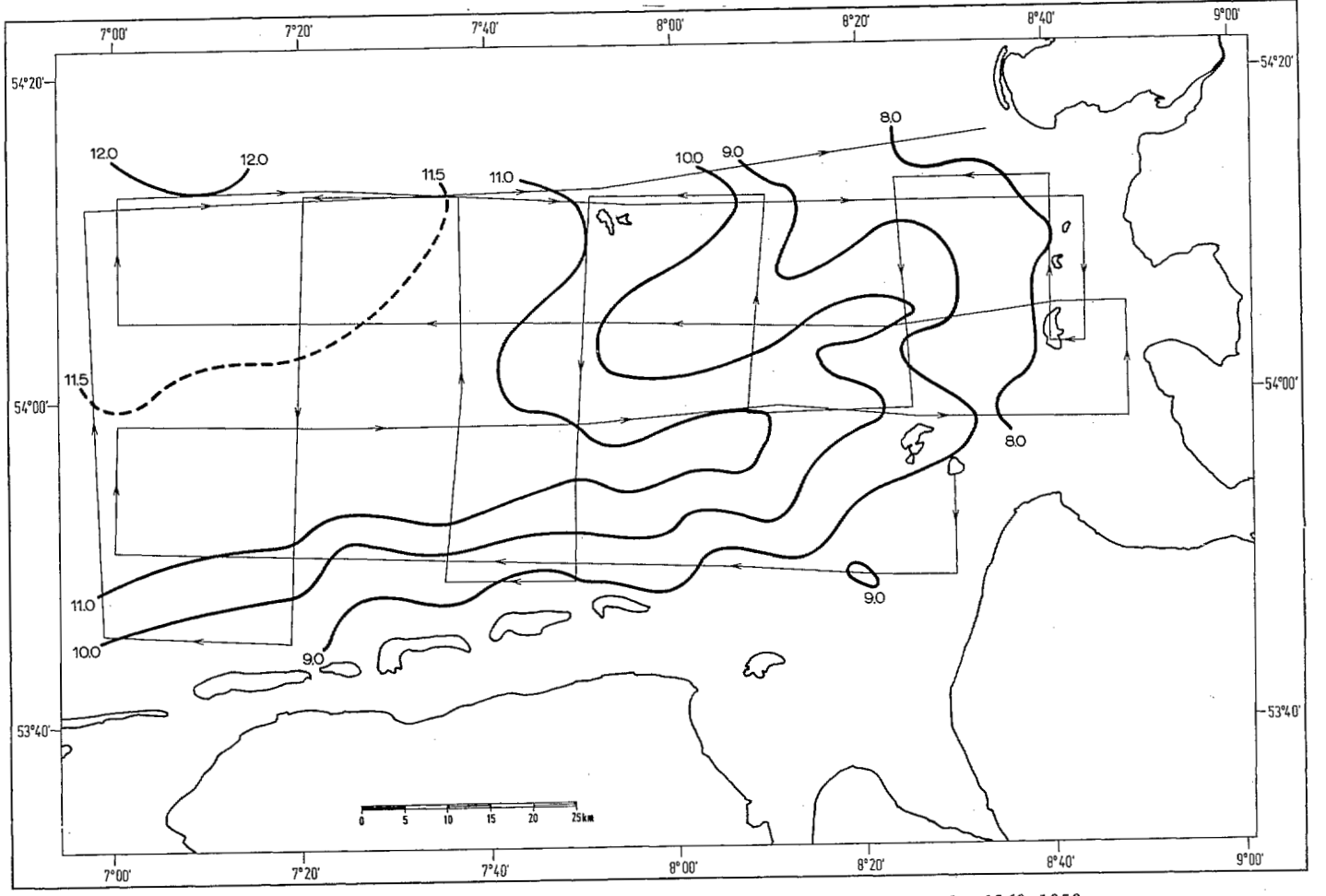


Figure 11 – *Surface temperature distribution of the Bay of Heligoland 11 November 1969, 1050 to 1410 hr CET. Surface temperature in °C. The lines with the arrowheads show the flight path*

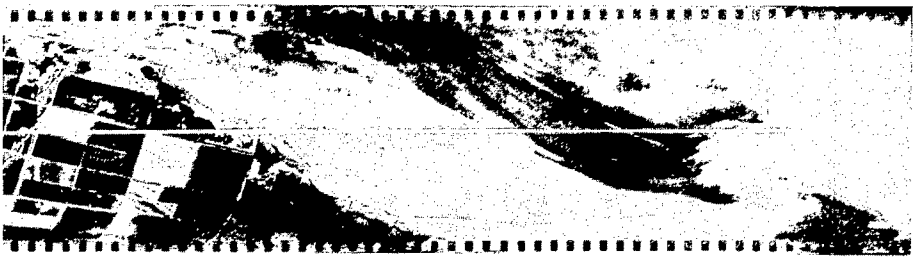


Figure 12 – *Infra-red photographic film-strip*

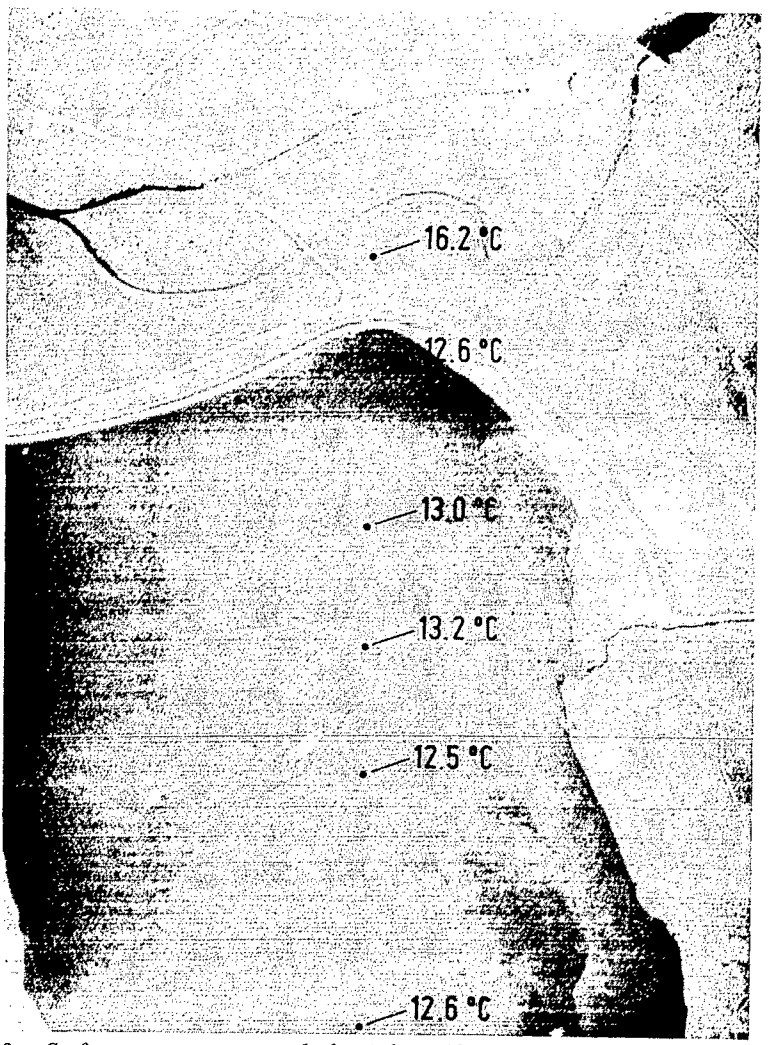


Figure 13 – *Surface temperature record of an infra-red line scanner system from the eastern part of the reservoir near Lechbruck on 20 August 1970, 1531 hr. Increasing density accords with decreasing temperature. Surface temperature values from synchronous IRT measurments. Line scanner record : Deutsche Forschungs– und Versuchsanstalt fuer Luft– und Raumfahrt, Institut fuer Flugfunk und Mikrowellen, Oberpfaffenhofen (G. Miosga). Released by Regierung von Oberbayern, No. 315/302*

In accordance with the season, the sea was cooling down. In the coastal shallow waters this effect was greater than in the deep sea because of the lower thermal capacity, so that temperature differences of 3°, at a distance of 50 km were produced. Satisfactory local resolution was obtained by the close spacing of the flight paths over the area. This shows the advantage of the airborne method in comparison with the traditional water surface temperature measurements which only yield the values of single points. A comparison between the airborne results and the water temperature measurements of light ships stationed in the target area is given in Table 3. It yields good agreement except in the case of the lightship P 12.

Table 3. *Comparison of the airborne temperature measurements of 11 November 1969 and the water temperatures reported from light ships*

Light ship, position			Airborne measurement		
			Time: 07 CET	19 CET	12-14 CET
P 8	54.3° N	7.2° E	11.8	11.8	12.1
P 12	54.0° N	7.9° E	12.4	12.1	10.5
Elbe 1	54.0° N	8.2° E	10.5	10.8	10.6
Weser	53.9° N	7.9° E	9.8	10.1	10.2

POSSIBLE FURTHER DEVELOPMENTS

The instrumental developments have only now been finished. In future the so-called line scanner systems will become important. They scan the area below the aircraft vertical to the flight path. Thus they yield not only results from a single line, but also from a relatively wide strip. This makes possible an uninterrupted cover of a ground or water area. The measurements are recorded on a photographic film strip by exposure to a lamp with a signal proportional to intensity. Thus a thermal image is obtained where the difference in density are differences of the surface temperature. Figures 12 and 13 show examples. In this case the temperature scale ranges from dark (low temperatures) to bright (high temperatures). The high resolution of the system is convincing. However, there is a restriction. The measurement is taken vertically from above only on the centre line of the strip. The angle decreases from 90° to the horizontal at the centre to 30° at the edges.

Thus the optical path length, and thereby absorption and emission of the air layer as well as the reflectivity of the water, changes. The appropriate corrections vary with the distance from the centre line (Saunders, 1967).

Since infra-red line scanners do not yet offer the high accuracy of the IRT described above, a combination of both types of instruments promises good results for the future.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Bolle, H. J. 1964. Ausarbeitung eines Verfahrens zur Vorhersage thermischer Strahlung. Die Abhängigkeit der Ortungsweite im infraroten Spektralbereich von meteorologischen Parametern. Bericht III zum Forschungsauftrag T-369-I-203 des BMVtdg. Meteor. Inst. Univ. München, vol. VIII, 168 p.

Clark, J. R.; Stone B. 1964. Use of the infrared thermometer in routine coastal survey. *Proc. O.N.R.–S.E.A.S. Workshop on Infrared Techn.*, U.S. Dept. of Interior, Washington, D.C.

Friedman, D. 1969. Infrared characteristics of ocean water (1.5-15 μ). *Appl. Optics,* vol. 8, p. 2073-2078.
Gaevskiy, V. L.; Rabinovich, Y.I.; Reshetnikow; A.I. 1965. Measurement of the water temperature of the Caspian Sea with a radiation thermometer. *Soviet Hydrol., Select. Pap.,* Washington D.C., no. 4, p. 402-405.
Grossman, R. L.; Bean B. R.; Marlatt W. E. 1969. Airborne infrared radiometer investigation of water surface temperature with and without an evaporation-retarding monomolecular layer. *J. Geophys. Res.* vol. 74 p. 2471-2476.
Kislovskii, L. D. 1959. Optical characteristics of water and ice in the infrared and radiowave regions of the spectrum. *Opt. i. Spectroskopiya,* vol. 7, p. 201-206.
Lorenz, D. 1966. The effect of the long-wave reflectivity of natural surfaces on surface temperature measurements using radiometers. *J. Appl. Meteor.,* vol. 5, p. 421-430.
—, 1967. Temperaturmessungen von Boden- und Wasseroberflächen von Luftfahrzeugen aus. *Pure Appl. Geophys.,* vol. 67, p. 197-220.
—, 1968. Temperature measurements of natural surfaces using infrared radiometers. *Appl. Optics,* vol. 7, p. 1705-1710.
—, 1969a. Der Einfluß einer Luftzwischenschicht auf meteorologische Oberflächentemperaturmessungen mit Radiometern. *Tellus,* vol. 21, p. 76-88.
—, 1969b. Flugzeugmessungen der Wasseroberflächentemperatur von Binnenseen. *Ann. Meteor.,* N. F., no. 4, p. 277-282.
—, 1970. Flugzeugmessungen der Oberflächentemperatur über See. Teil 2 d. Abschlußberichts zum Forschungsauftrag T/0230/92350/91326 des BMVtdg. Dt. Wetterd. Offenbach a. M., 45 p.
Ragotzkie, R. A.; Bratnick, M. 1966. Infrared temperature patterns on lake Superior and inferred vertical motions. Univ. of Wisconsin, Dept. of Meteor. Techn. Report No. 27, Madison, Wisc.
Richards, T. L. 1966. Great Lakes water temperatures by aerial survey. Publ. Union Geodes. Geophys. Internat. Assoc. Internat. Hydrol. Sci., no. 70, p. 406-419.
—, Irbe, J. B.; Massey, D. B. 1969. Aerial surveys of Great Lakes water temperatures April, 1966 to March, 1968. Dept. of Transport, Meteor. Branch. Climatological Studies, no. 14. Toronto, 55 p.
Saunders, P. M. 1967. Aerial measurement of sea surface temperature in the infrared. *J. Geophys. Res.,* vol. 72, p. 4109-4117.
—, Wilkins, C. H. 1966. Precise airborne radiation thermometry. *Proc. 4th Sypos. Remote Sens. of Envir.,* Ann Arbor, Mich., p. 815-826.
Stommel, H. 1963. Rapid aerial survey of the Gulf Stream with camera and radiation thermometer. *Science,* vol. 117, p. 639-640.

DISCUSSION

Günneberg: Localization of the spectral region results in the fact that the correction required for reflection is considerably smaller. On the other hand, this also reduces the sensitivity of your instrument. We at the Federal Institute of Hydrology need the exact water surface temperature for measuring the heat exchange because the daily fluctuations of the water temperature of large rivers amounts only to a few tenths of one degree. Instruments with a noise equivalent of less than 1/10° can be easily procured. Messrs. Raimann offered us one with a noise equivalent of 1/100°, for DM 3,000. In such cases, however, the correction required in consequence of the reflected portion of the sky light is much larger than the measuring sensitivity, and we can no longer risk calculating with the aid of the correction diagram. We have used a practical remedy that is well suited to quiet water surfaces. We mounted an aluminium mirror at the instrument and arranged it so that the spot observed by the instrument reflected not the sky light but, instead, the mirror itself, and this mirror then reflected the water surface onto itself. As a result, we obtain zero correction because, in turn, each reflected portion corresponds exactly to the true water temperature.

Do you have a more practical arrangement which would allow measurements also in the case of a rough water surface ?

Lorenz: The localization of the spectral region within 9.5 to 11.5° μm is no longer a problem as far as the technical aspect is concerned. Instruments are now available which, with this localization of the spectral region, allow stationary measurements to be carried out at a relative measuring accuracy at which temperature differences of 0.1° can be ascertained without difficulty. Measurement is then possible within the narrow range of this spectral region with a minimum of correction. The influence of the long-wave sky radiation, however, is still noticeable. The method of using a mirror is a makeshift arrangement that can be used for stationary measurements, but which may also exert an influence on surface temperature, if only by shading the water. By using a second instrument, one could measure the long-wave sky radiation at a considerably reduced sensitivity level and at a defined aperture angle corresponding to the reflection of the given roughness of the water surface.

Günneberg: Referring to the question of shading: if you have an instrument with a 7° angle of view and then, for reasons of safety, you use a larger mirror having a diameter of about 15°, this will still be a shading of the water surface and will effect a modification in heat exchange of 10 per cent at the most.

And now the question of the second instrument: naturally, at first we always also sighted the sky with our instrument at certain intervals, but this requires change-over to another measuring range because of the entirely different temperature.

Actually, a second instrument of lower quality must be connected in parallel, and then the two signals must be substracted electrically, one from the other.

Hinrich: 1. Has the influence of the wind on the change of water temperature due to the warm water being driven off already been investigated ?

2. Is the wave motion of any influence, in particular in connexion with the formation of foam ?

3. Does the measurement of surface temperature as such already allow a forecast of icing conditions?

Lorenz: The influence of the waves: we always measure the surface temperature and this means that, in the case of waves, we obtain the surface temperature on the wave crest as well as in the wave trough. If the surface temperature changes as a result of any influence exerted by the wind, this will, of course, be included in the measurement.

So far, we have not been concerned with the forecasting of icing conditions.

Earth satellites and their applications in hydrometry and hydrology

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Abstract. Earth satellites can be used for operations or research in hydrometry and hydrology in two capacities: (1) as a platform for remote sensors, and (2) as a data relay device. Whether the spacecraft are manned or not, two types of orbits are generally the most useful: (1) sun-synchronous, which are relatively low altitude and near-polar inclination; and (2) earth-synchronous, which are relatively high and near-equatorial. The types of remote sensing equipment that are available or under development range across the electromagnetic spectrum from microwave through infra-red and visible frequencies. There have already been limited applications of visible and infra-red data from meteorological satellites to hydrologic and related problems. More extensive experiments are planned in connexion with the upcoming meteorological and earth resources survey satellites. Equipment is being developed that will enable analogue or digital output from hydrometric or hydrologic observation systems at the ground to be relayed via satellite to central data processing centres.

LES SATELLITES TERRESTRES ET LEURS APPLICATIONS EN HYDROMETRIE ET EN HYDROLOGIE

Résumé. Les satellites terrestres peuvent être utilisés pour des opérations ou des recherches en hydrométrie et en hydrologie de deux façons : comme plates-formes portant des capteurs à distance ; comme relais de données.

Selon que l'engin spatial comprend un équipage humain ou non, deux types d'orbite sont en général utilisés : orbites synchrones du Soleil, dont l'altitude est relativement basse et dont l'inclinaison voisine des pôles ; orbites synchrones de la Terre, dont l'altitude est relativement élevée et dont l'inclinaison est proche de l'équateur. Les types d'équipement des capteurs à distance qui sont disponibles ou en cours d'étude couvrent tout le spectre électromagnétique, depuis les ondes ultra-courtes jusqu'aux fréquences du spectre infrarouge et visible. Certains problèmes hydrologiques et d'autres, connexes, ont déjà bénéficié de quelques applications de transmission par le spectre visible et infrarouge à partir de satellites météorologiques. Des expériences plus complètes sont projetées en liaison avec les prochains satellites destinés à l'information météorologique et à l'étude des ressources terrestres. On étudie des équipements qui permettront aux données analogiques ou numériques provenant de systèmes d'observations hydrométriques ou hydrologiques au sol d'être relayées par satellite vers un centre de traitement de données.

UTILIZACIÓN DE SATÉLITES EN HIDROMETRÍA E HIDROLOGÍA

Resumen. Los satélites se pueden utilizar en operaciones o investigaciones de hidrometría e hidrología para dos fines: (1) como un receptor para detectores remotos y (2) como un dispositivo de intercambio de datos. Sean tripuladas o no las naves espaciales, los dos tipos de órbitas que presentan mayor utilidad son: (1) síncromas al sol, que son relativamente de baja altitud y de inclinación casi polar; y (2) síncromas a la tierra, que son relativamente altas y casi ecuatoriales. Los tipos de equipo de detección a distancia que se encuentran disponibles o en desarrollo trabajan a través del espectro electromagnético desde las microondas a las frecuencias infrarrojas y visibles. Hasta ahora sólo han existido aplicaciones limitadas de datos visibles e infrarrojos procedentes de satélites meterorológicos a los problemas hidrológicos. Se han proyectado experimentos de mayor extensión relacionados con los datos de los satélites meteorológicos para estudio de recursos terrestres. Se está desarrollando un equipo que permita que los datos analógicos digitales procedentes de los sistemas de observación hidrométrica o hidrológica se transmitan, por medio de satélites, a los centros principales de proceso de datos.

ЗЕМНЫЕ СПУТНИКИ И ИХ ПРИМЕНЕНИЕ В ГИДРОМЕТРИИ И ГИДРОЛОГИИ

Резюме. Земные спутники могут быть использованы для проведения наблюдений или исследований в гидрометрии и гидрологии в двух направлениях: 1) как платформа для дистанционных датчиков и 2) как устройства для передачи информации.

Независимо от того, управляемы космические корабли или нет, наиболее успешными являются, как правило, два типа орбит: 1) синхронная с солнечной, которая имеет относительно низкую высоту и близкое к полюсному наклонение и 2) синхронная с земной, которая сравнительно высока и почти экваториальна.

Типы дистанционного чувствительного оборудования, которые имеются в наличии или разрабатываются в насоящее время, чувствительны к прохождению микроволн инфракрасных и видимых частот электромагнитного спектра.

Уже имеется некоторый опыт применения видимых и инфракрасных данных, получаемых от метеорологических спутников для гидрологических и связанных с ними проблем.

Более обширные экесперименты запланированы в связи с новейшими спутниками обследования метеорологических и земных источников.

Оборудование совершенствуется с таким расчетом, чтобы быть способным передавать аналоговые и цифровые результаты от гидрометрических или гидрологических систем наблюдения на земле через спутники в главные центры обработки информации.

INTRODUCTION

Hydrometrists and hydrologists did not recognize as early as meteorologists the great potential for spacecraft sensing or measuring factors of interest to them. Therefore, meteorologists made the demands which set the designs for current remote sensing from satellites. Fortunately, the meteorologist's interests are in the same sphere as the hydrologist's and the meteorologist's choice of satellite orbits, polar sun-synchronous and earth-synchronous, are those most useful for hydrometry and hydrology. Because meteorological criteria for sensing, such as the portion of electromagnetic spectrum, are not those optimum for Earth's surface reflectances, current applications of spacecraft-observed data to hydrology, and especially to hydrometry, are very limited. Future Earth resources survey or environmental satellites should do much to improve this situation.

ORBITS, SATELLITES, AND SENSORS

Need for repetitive, daily global coverage has largely dictated the requirements for current meteorological satellite orbits. This requirement has resulted in the use of two types of orbits: (1) sun-synchronous, which is relatively low altitude and near polar in inclination; and (2) earth-synchronous, which is relatively high and is near-equatorial in inclination.

Current United States satellites with sun-synchronous orbits are ESSA, I-TOS and Nimbus and, by 1972, the Earth resources technology satellites (ERTS). This orbit permits the sensors from a single satellite to view any particular point on earth twice each day, once during daylight and once at night. Sensors carried on these satellites

are generally vidicon cameras and radiometric scanners. Vidicon camera systems, such as on ESSA 8 and 9, are useful only during the daylight portion of the orbit. These cameras are sensitive to broadbands of the electromagnetic (EM) spectrum, 0.5 μm to 0.7 μm. The automatic picture cameras transmit their information continuously, whereas the advanced vidicon camera system stores its data on magnetic tape, transmitting them upon command to a ground station. The data received by the ground station is retransmitted to a computer centre where the analogue signal is digitized before it is gridded, normalized for solar illumination, and then displayed as a photographic print or digitized print-out from the computer.

The I-TOS series, such as the now orbiting I-TOS 1, observes the Earth's surface by automatic picture camera and the advanced vidicon camera system, as well as by means of a scanning radiometer. The scanning radiometer data can be stored and transmitted upon command of a ground station's signal and also broadcasts for local reception by slightly modified automatic picture receivers. This sensor, which is sensitive to infra-red radiation in the 10.5-12.5 μm ranges, has night as well as daytime capability. The scanning radiometer's EM bandwidth used for daylight viewing is the same as for current vidicon cameras.

During 1969 a successful testing was made from Nimbus III of a satellite infra-red spectrometer. Smith and Woolf (in press) showed that atmospheric temperature profiles made from such data were quite comparable with those obtained by standard meteorological instruments. Temperature and water vapour sounders, such as the satellite infra-red spectrometer, hold much promise for improvement of quantitative precipitation forecasts. These improvements will be realized through the increased number of soundings for evaluating the mass and motion structure of the atmosphere. Improvement of quantitative precipitation forecasts would be of great importance, since this should lead directly to improving the forecast of watershed runoff.

Due to the sensors' field of view and resolution requirements on ERTS, the viewed areas will not overlap on successive orbits. Furthermore, repetitive orbits of an ERTS will, in general, be only once in eighteen days and contiguous viewing orbits will be a day apart. Earth scientists should expect to find ERTS data more useful than those from current meteorological satellites because of (a) increased resolution and (b) multispectral capability. Resolution will be about 100 m during the day and about 200 m at night. Multispectral capability is to be obtained by the use of three return-beam vidicons viewing, separately, 0.3-0.5, 0.5-0.6, 0.6-0.7 μm bandwidths. ERTS's scanning radiometer will have the same capability in the visible bands and, in addition, in the near (about 1.0 μm) and far (10-14 μm) infra-red.

Earth-synchronous orbits are in the equatorial plane at a distance of 36,000 km above the Earth's surface. There are two NASA Applications Technology Satellites (ATS) of hydrological interest that observe the Earth from such orbits. Disadvantages of this type of orbit are that it takes four satellites to view the entire Earth and that the area near the poles is poorly observed. A good attribute of this type of orbit is that it permits the sensors to view about one-fourth of the globe constantly, or to monitor continually any *in situ* measurements that are telemetered from the same portion of the globe. ATS spin-scan cameras view the Earth, from pole to pole, once every twenty minutes at the same wave-length and resolution as the polar-orbiting meteorological satellites. Thus the ATS can provide relatively short-interval time sequences for part of the globe to supplement the once-a-day viewing of lower altitude, polar-orbiting satellites. At present, there is no night-time viewing capability from these satellites.

The Geostationary Operational Environmental Satellite (GOES), planned for 1972, is earth-synchronous and will have day and night observation capability.

Furthermore, this satellite should be able to collect data from a variety of *in situ* automatic observing platforms and to broadcast specialized products to remote areas. Local users will be able to receive and display this information with relatively simple ground stations.

The first Apollo Applications Programme manned Saturn Workshop (Sky Lab) is planned for 1972 or 1973. These spacecraft (near 50° inclination and about 410 km orbit) will carry equipment, probably consisting of a six-camera cluster for multispectral, hard-film recovery, for a limited Earth survey experiment. Future equipment is likely to include Earth survey sensors whose size, weight, power, or antenna requirements are too stringent for ERTS or other small, unmanned spacecraft.

HYDROMETRY FROM SPACECRAFT

Inferred or direct hydrometric measurements from satellites appear to be very limited, now or in the future. The greatest use for satellites in hydrometry should be in the telemetering of relatively inaccessible ground measurements to a central data processing centre.

Probably the only current satellite remote measurements applicable to hydrometry are in the far infra-red, 10-14 μm bandwidth. Measurements of energy return in this bandwidth, along with a known or assumed emissivity, permits an estimate of the average thermal temperature of the viewed area to be made. Other considerations, such as atmospheric transmission loss, intervening clouds between sensor and area of interest (target), error in assigning emissivity value to target, and ground spot resolution, present problems to be overcome in this method of measuring surface temperature. The smallest spot resolution of a current satellite sensor in the far infra-red is approximately 4 nautical miles on Nimbus III and I-TOS 1. Theoretically then, this is the smallest area for which a surface temperature could be measured, but due to the problems mentioned above, the smallest measurable area in practice is some multiple of the minimum spot resolution. Smith *et al.* (1970) showed that, by combining Nimbus HRIR scan spot radiances for a small area into a histogram, the brightness temperature associated with the model radiances is the statistically most probable sea-surface temperature.

In parts of the microwave region of the EM spectrum it will be possible to measure radiation upwelling from Earth through cloudiness. Also, by using several polarizations and measuring energy return from various look angles it will be possible to make some evaluation of the viewed surface's emissivity or temperature. Antennae necessary to make spot resolution comparable with that achievable in the infra-red region are prohibitive from weight and size considerations on currently planned satellites. Manned space stations, such as those planned with the Sky Labs and which are expected to have erectable and directable large antennae, may make feasible relatively high-resolution microwave sensing.

Utilizing a satellite as a communications link between ground-based sensors and a data processing centre is a concept that will have wide applications. Such measurements as water flow, water temperature, water quality (O_2, NO_2, etc.), snow depth or water equivalent, and soil moisture can be made *in situ* and then relayed by a relatively inexpensive transmitter to an orbiting satellite, thence retransmitted to a central collection point. A successful experiment along these lines was conducted by using a single earth-synchronous satellite, the ATS I, to collect and transmit precipitation and stream gauge data (Flanders *et al.*, 1969). Thus satellites of the early and mid-seventies that have telemetering relay capabilities may prove to be a most useful adjunct to the fiel of hydrometry.

HYDROLOGY FROM SPACECRAFT

The meteorological satellite's visible spectrum band, 0.5 to 0.7 μm, is not optimum for viewing Earth surface reflectances. Studies by Chapurskiy *et al.* (1968) and Colwell (1966) indicate that a bandwidth from 0.7 to 0.9 μm would be more useful, but even more desirable would be measurements at several wave-lengths from visible and near infra-red regions of the EM spectrum. Colwell (1966) demonstrated this when he made comparisons of aircraft data taken in three different wavebands to identify, among other features: (1) saturated, moist and dry meadows, (2) origin or a spring, (3) stream of clear water, and (4) irrigated areas. Colwell used photographic film, but a video camera or a scanning radiometer operating in these three wave-lengths is feasible also. It is necessary, of course, that the resolution in the three bandwiths be identical and that exact registration among bandwidths be obtained.

Barnes and Bowley (1968) using photographic interpretation techniques, and McClain and Baker (1969) using digitized data, have shown the application of satellite data to delineating areal snow cover. Comparison of Figure 1 and Figure 2, which are digitized mosaics from ESSA 9 AVCS[1], illustrates photographic interpretation of snow areas.

An ice lead in Hudson Bay can be seen in Figure 1, and its indistinct but definite pattern is confirmed in Figure 2. The feathery appearing streaks over the Hudson Bay, which partially obscure the ice lead as well, are typical cloud patterns. White Sands, New Mexico (33° N, 106° W) is evident in Figure 1, but its absence in Figure 2 is evidence that clouds are present in that region. Lack of clouds is shown by recognizable features such as the dendritic pattern associated with mountain snow, a pattern that is especially well marked in the Canadian coastal range and in the Rocky Mountains of Canada and the United States (see Figures 1 and 2).

Digitized data application is shown in Figure 3. This is a composite over a five-day period, which includes the days given in Figures 1 and 2, of the minimum brightness value retained for any geographical matrix point defined in the computer. Since cloud positions from day to day are transitory relative to snow, this five-day compositing of minimum brightness will essentially delineate the snow or ice fields, except in cases of persistent cloudiness. An example of cloud suppression achieved by the composite minimum brightness technique is shown by comparing the Gulf States in Figures 1, 2 and 3. No clouds appear in this area on the five-day composite chart whereas clouds do appear on the daily digitized mosaics.

CONCLUDING REMARKS

Direct applications of current satellite data to hydrometry and hydrology are limited to surface temperature evaluation (such as over large snow or ice fields, large lakes and land areas) and to areal delineation of snow or ice fields. Of these, only the areal snow delineation is currently performed operationally. It is expected that by 1971 earth surface temperatures obtained by I-TOS satellites will be routinely available from ESSA's satellite centre. With the advent of a very high resolution radiometer in the far infra-red on I-TOS and the lower orbiting ERTS's scanning radiometer, this temperature analysis could be extended on a limited basis to ice delineation, evaporation, evapotranspiration, or floods in connexion with rivers, small lakes,

1. These figures are made by compositing digitized data from rectified, normalized for solar illumination, vignette-corrected, individual frames.

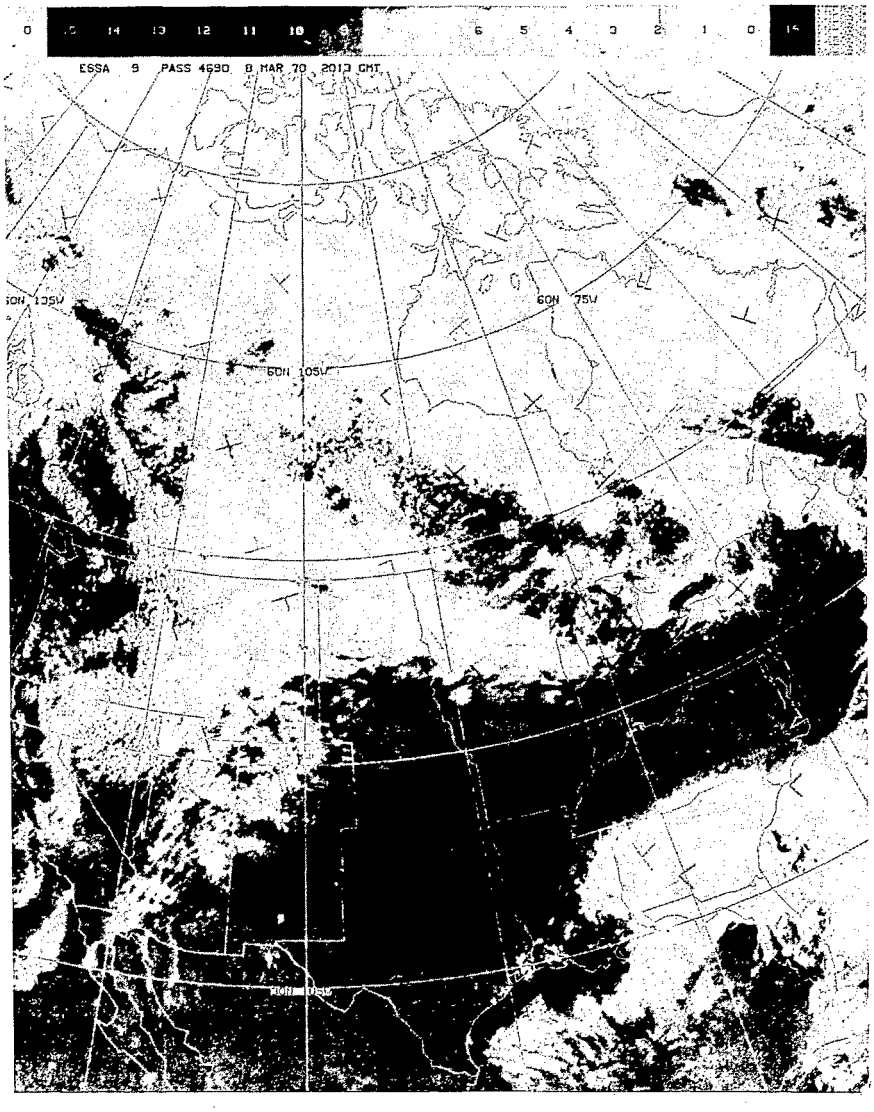


Figure 1 – *ESSA-9 digitized mosaic for 8 March 1970*

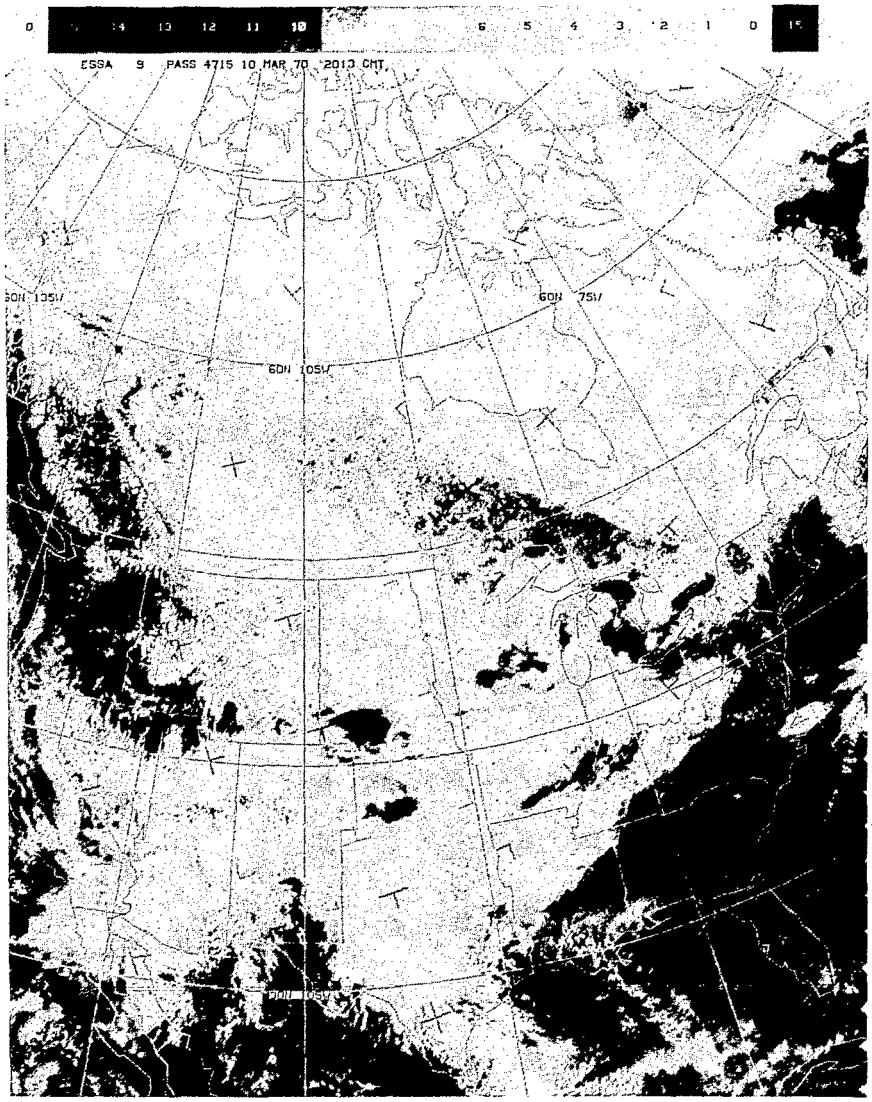


Figure 2 – *ESSA-9 digitized mosaic for 10 March 1970*

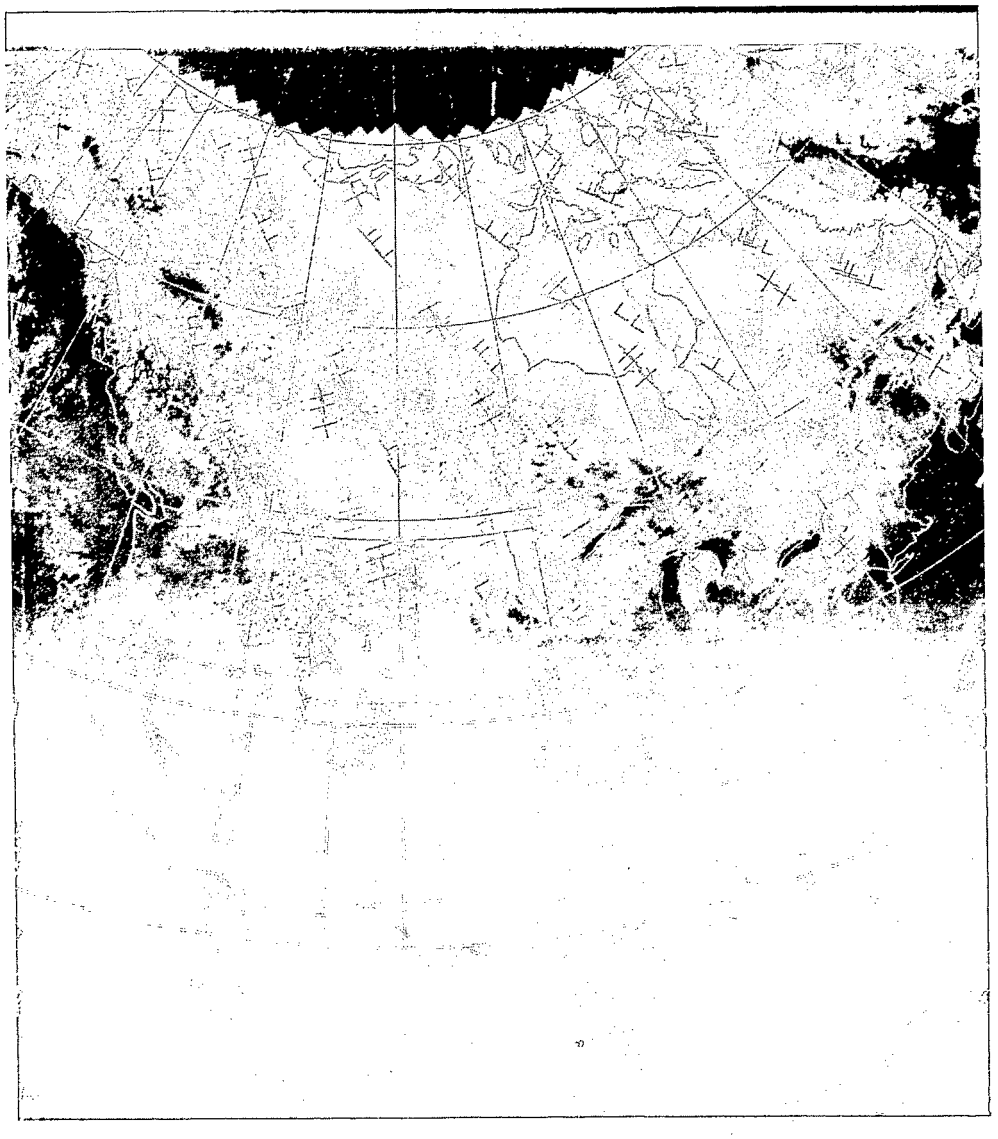


Figure 3 – *Five-day minimum brightness composite for 6 to 10 March 1970*

reservoirs, and land areas. With the proposed multispectral capability on ERTS, reflectances from surface features will become available that should be more useful for hydrological purposes. By mapping the radiances, targets of interest should often be detected and, within spatial resolution capabilities, their areal extent determined. Furthermore, photo-interpretation techniques involving such factors as pattern recognition, textural variations, etc. can often yield additional information. Weak or ambiguous interpretations based on spectral signatures alone can often be strengthened or resolved by bringing to bear physical, geographical, or other constraints.

Because of economic, technical, or practical reasons, some hydrometrical or hydrological measurements will not or cannot be obtained remotely, and there will be a need for *in situ* measurements. Here the satellite offers a means of relaying data from a remotely located site, where poor or no communications exist, to a point where data are processed or used. Furthermore, processed data could be retransmitted for local use. Even now plans are under way in the United States for employment of such a system.

Temperature and water vapour sounders on board satellites hold much promise for improvement of quantitative precipitation forecasting, which should lead directly to improved forecasts of watershed runoff.

In the near future the greatest concern may not be that enough useful data are collected, but how the enormous amounts of data collected will be handled and their information content determined.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Barnes, J.C.; Bowley, C. J. 1968. Operational Guide for Mapping Snow Cover from Satellite Photography. ESSA Contract no. 2-162-67(N), Allied Research Associates, Concern, Massachusetts. 116 p.

Chapurskiy, L. I.; Klemin, V. V.; Andreyeva, N. I.; Startseva, M. V. 1968. Spectral brightness of clouds and landscape objects in the visible and near infrared sectors of the spectrum. *Sputnikovya Meteorologiya, G.G.O. Trudy,* vol. 221, Hydrometeorological Press, Leningrad. NASA Technical Translation (TT-F 589), p. 225-239.

Colwell, R. N. 1966. Uses and limitations of multispectral remote sensing. *Proceedings of the Fourth Symposium of Remote Sensing Environment,* University of Michigan, Ann. Arbor, Michigan, p. 77-100.

Flanders, A. F.; Kohl F. V.; Davis, T. W. 1969. Hydrologic communications experiment on the Applications Technological Satellite (ATS-1). *Summaries of the Sixth Symposium on Remote Sensing,* University of Michigan, Ann. Arbor, Michigan, p. 12-17.

McLain, E. P.; Baker, D. R. 1969. Experimental large scale snow and ice mapping with composite minimum brightness charts. *National Environmental Satellite Center Technical Memorandum NESCTM-12,* Environmental Science Services Administration, Washington, D.C., 19 p.

Smith, W. L.; Rao, P. K.; Koffler, R.; Curtis, W. R. 1970. The determination of sea-surface temperature from satellite high resolution infrared window radiation measurements. Submitted to *Journal of Physical Oceanography.*

Woolf, H. M. In press. Regression method for obtaining real time temperature and geopotential height profiles from satellite spectrometer measurements and its application to SIRS observations. *Monthly Weather Review* (in press).

DISCUSSION

McDowall: I should like to provide some supplementary information regarding the Lake Ontario surface temperatures which were acquired by Canada as "ground truth" data for comparison with the satellite observation made by the United States and just presented.

The Canadian Meteorological Service compared Airborne Radiation Thermometer (ART) observations with surface water temperature (bucket) measurements for a variety of weather conditions. A rapid technique was developed to correct far atmospheric effects. These feasibility studies show that ART observations could be made of water surface temperature to within 1°C using the Barnes PRT5 radiation thermometer from altitudes of about 1 000 m. Corrections for reflection effects were not necessary to achieve the ± 1°C accuracy required for several water resources and mangement purposes. The programme is more economical than maintaining surface stations.

As a result of these satisfactory studies, Canada established ART flights over several lakes on a routine basis. During the past year, flights were conducted over the whole of Lake Ontario in every month. About ten north to south flight lines were used so that the whole lake was covered in about 3-5 hr. During the flights and several times per hour, the radiometer performance was checked within the aircraft. The readings were made on a chart recorder. The charts were analysed, corrected for atmospheric absorption and finally presented in the form of isotherms on a map of the lake. The ART "ground truth" data shown by Mr. Baker was obtained by Canada in the above manner.

XI Recording and teletransmission of data

Enregistrement et télétransmission des mesures

Système de télélimnimétrie LGR

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Résumé. Les possibilités d'exploitation du réseau téléphonique public, tant pour la transmission d'informations que comme source d'énergie, ont permis le développement d'équipements de télélimnigraphie économiques, simples et adaptables à d'autres mesures hydrologiques et météorologiques.

Le système comporte un poste de commande et des stations de mesure. Le premier forme automatiquement et à heures fixes les numéros d'appel des différentes stations de mesure et transforme les signaux reçus en valeurs numériques, corrigées en fonction du télé-étalonnage de chaque mesure.

L'énergie utilisée étant prélevée au réseau téléphonique, il en résulte une simplification considérable des stations de mesure, qui s'accommodent de conditions quelconques du site rural.

L'ensemble est en fonctionnement depuis plus d'un an et fournit des données sur bande perforée, apte à tout traitement ultérieur par ordinateur.

L'investissement d'installation est particulièrement réduit en regard des autres systèmes existants.

Ce système permet de limiter d'une façon considérable les interventions humaines.

LGR, A SYSTEM FOR TELELIMNIMETRY

Abstract. The possibilities of utilization of the telephone network for data transmission, as well as a power source, have allowed the development of simple and economical teletransmission devices for the measurement of water stages and other hydrological and meterorological data.

The system consists of a main monitoring station and a set of measuring stations. The former automatically dials and calls the individual measuring stations at predetermined times, calibrates each signal and converts it into numerical values.

Since the power used is that of the telephone line, the measuring stations are considerably simplified and can be adapted to any conditions required in the field.

The system has been operative for more than a year. The data are punched on tape and, therefore, can be fed directly to the computer for calculation.

Compared with others, this system allows a drastic reduction of the initial investment and the minimum of human intervention.

SISTEMA DE TELEMEDIDA

Resumen. Las posibilidades de explotación de la red telefónica pública, tanto para la transmisión de informaciones como para fuente de energía, han permitido el desarrollo de equipos de telelimnigrafía económicos, sencillos y adaptables a otras medidas hidrológicas y meteorológicas.

El sistema está compuesto de un puesto de mando y de estaciones de medida. El primero prepara automáticamente y a horas fijas los números de llamada de las distintas estaciones de medida y transforma las señales recibidas en valores numéricos, calibrando cada señal. Puesto que la energía utilizada procede de la red de teléfonos se obtiene una considerable simplificación de las estaciones de medida, que se acomodan a las condiciones existentes.

El conjunto está en funcionamiento desde hace más de un año y proporciona datos en cinta perforada, apta para cualquier tratamiento posterior por ordenador.

1. Les auteurs expriment leur gratitude à la firme MBLE et à l'IRSIA pour leur soutien moral et financier ainsi qu'à MM. W. Trappeniers et J. Riguelle pour leur aide précieuse tout au long de la mise au point du système LGR.

La inversión de instalación es particularmente reducida, con respecto a otros sistemas existentes. El sistema permite una independencia total a la intervención humana.

ТЕЛЕМЕТРИЧЕСКАЯ СИСТЕМА ДЛЯ ПЕРЕДАЧИ ДАННЫХ ОБ УРОВНЕ ВОДЫ И ДРУГИХ ЭЛЕМЕИТОВ

Резюме. Возможности использования телефонной сети для передачи данных, а также источников энергии сделали возможным совершенствование простых и экономичных телепередающих устройств для измерения уровней воды и других гидрологических и метеорологических данных.

Система состоит из главного монитора и ряда измерительных станций. Монитор набирает монер и автоматически вызывает отдельные измерительные станции в заранее установленное время, калибрует каждый сигнал и преобразует его в цифровые величины.

Поскольку используемая энергия есть та, которая принадлежит телефонной линии, измерительные станции значительно упрощены и могут быть приспособлены к любым полевым условиям.

Система работает больше года: информанция наносится на ленту и поэтому может быть введена непосредственно в компьютер для вычислений.

По сравнению с другими эта система дает резкое уменьшение первоначального капиталовложения и полную независимотсь от человека.

La télétransmission de données se généralise dans l'industrie par suite d'un concours de circonstances techniques, sociales et économiques. Il n'en est pas de même en ce qui concerne la collecte et le dépouillement des données en sites naturels. Cette situation, fort regrettable en matière de mise en valeur des ressources naturelles, provient du coût des systèmes de télémesure disponibles ainsi que de leur peu d'adaptation à un essaimage en milieu rural et à la rusticité des sites de prélèvement de données.

Les systèmes de télémesure existants nécessitent généralement des lignes téléphoniques permanentes non alimentées; leur adaptation à l'environnement rural demande de lourdes infrastructures ainsi qu'un entretien astreignant. Aussi faut-il se contenter actuellement soit de lectures directes avec inscriptions intermittentes par des agents préposés, soit de systèmes enregistreurs. Les nombreuses interventions exigées par le traitement de ces données introduisent toutes sortes d'erreurs, sans compter une perte de temps considérable. En outre, la distribution des sites de prélèvement sur un territoire est actuellement par trop limitée. Or la conduite des études et des recherches en hydrologie, en météorologie, en agronomie, etc., consiste en une succession de prélèvements de données à des fréquences variables en de nombreux endroits d'accès souvent difficile. En conséquence, le système de télémesure à adopter doit réaliser automatiquement l'alimentation de nombreuses stations de mesure peu encombrantes et peu coûteuses, la centralisation de données précises en vue d'un traitement rapide par ordinateur et le contrôle à distance du fonctionnement correct des appareils. L'utilisation du réseau téléphonique public existant permet de répondre favorablement à ces critères d'alimentation, de transmission et d'exploitation.

Un tel système a été conçu et est opérationnel au Laboratoire du génie rural de l'Université catholique de Louvain. La simplicité, la fiabilité, la précision, la robustesse et l'économie obtenue à l'investissement rendent le système approprié à toute mesure à distance et plus particulièrement en sites naturels.

La mise au point d'un dispositif pour le contrôle de la séquence des opérations ou "décrocheur-raccrocheur" LGR (brevet déposé) a permis la réalisation d'un système dont la description et le fonctionnement sont décrits ci-après.

Des informations plus techniques sont fournies en annexes.

LE SYSTEME DE TELEMESURE LGR

Deux ensembles distincts, le "poste chef" et la "station de mesure" sont reliés par une des lignes téléphoniques du réseau public. Lorsqu'une station de mesure est appelée manuellement ou lorsque le poste chef la sollicite automatiquement à heures déterminées, elle est alimentée par la ligne téléphonique pendant la durée d'une communication, car elle ne possède aucune autre source d'énergie (telle que batteries ou alimentation en basse tension). En effet, la ligne téléphonique fournit une alimentation sous forme de courant continu et réalise le support nécessaire à la transmission des informations sous forme de courant alternatif.

L'élément senseur ou capteur pour le prélèvement de l'information est choisi en fonction du type de mesure. Un potentiomètre convient très bien comme capteur en télélimnimétrie et permet, par sa simplicité et son faible encombrement, de réduire l'infrastructure au niveau des points de mesure (puits, piézomètre, pluviomètre, etc.).

Le poste chef reçoit les informations sous forme de fréquences et les achemine avec des références à une perforatrice de bandes. Ces références identifient la station par le numéro de téléphone et donnent l'instant de l'appel.

LA STATION DE MESURE

Chaque station se compose d'un certain nombre de capteurs, d'un décrocheur-raccrocheur et d'un oscillateur qui émet successivement les fréquences correspondant à chaque capteur (annexe 1).

Le décrocheur-raccrocheur et l'oscillateur sont généralement groupés dans un boîtier séparé du ou des capteurs. Les conducteurs nécessaires au raccordement entre capteurs et boîtier de même que ce dernier peuvent être enterrés ou disposés en aérien selon les exigences du site.

Le décrocheur-raccrocheur a comme fonction principale la commande automatique (i_1 à la fig. 1) de mise en service et d'arrêt de l'oscillateur qu'il relie à la ligne téléphonique au reçu d'un appel en provenance du poste chef et qu'il libère dès que l'information a été transmise. De plus, il permet de relier successivement plusieurs capteurs au même oscillateur et d'effectuer le télé-étalonnage, ainsi qu'on le verra ci-après.

L'énergie d'activation du décrocheur-raccrocheur provient du courant alternatif de sonnerie, mais le courant de la batterie centrale du réseau téléphonique peut intervenir dans la suite des opérations. Les détails de fonctionnement et un schéma sont donnés à l'annexe 3.

Oscillateur (fig. 1). La fréquence est générée par un multivibrateur astable T_1, T_2 qui sert d'oscillateur et l'on montre à l'annexe 2 qu'elle varie d'une façon strictement linéaire en fonction de la position du curseur du potentiomètre P. Dans le cas d'une mesure limnimétrique, la position du curseur est liée à celle d'un flotteur par exemple.

L'étage séparateur T_3 évite toute réaction directe de la ligne sur l'oscillateur et permet d'ajuster la valeur de la tension alternative de sortie qui est envoyée en ligne à travers le transformateur *Tr*.

Télé-étalonnage. Etant donné la linéarité du fonctionnement d'un tel oscillateur, l'étalonnage peut se réduire à la détermination des fréquences correspondant aux deux extrémités de la résistance *P*, puisqu'on sait après calibration quels niveaux y correspondent. Calibration et étalonnage sont effectués par téléphone. La calibration est réalisée à l'occasion de l'installation du point de mesure. Le télé-étalonnage, lui, est répété automatiquement et parallèlement à chaque mesure. Il supprime radicalement les glissements de l'étalonnage dus aux variations d'humidité et de température et au vieillissement des composants électroniques. Il permet, en outre, la surveillance permanente du bon fonctionnement des stations de mesure.

En garantissant une haute précision et la fiabilité du système, le télé-étalonnage permet aussi de réduire considérablement le prix des appareils, car il rend l'usage de composants coûteux superflu et évite la nécessité d'un réglage précis.

Le poste chef

Le poste chef est le dispositif automatique qui interroge les différentes stations à heures déterminées et qui enregistre les données de chaque point de mesure ainsi que les références. Il se compose principalement d'une horloge, d'un magnétophone, de deux fréquencemètres, de circuits logiques et d'une perforatrice de bandes.

L'horloge électrique synchronisée sert à déclencher la séquence d'appels des stations de mesure aux heures prévues. Elle a une réserve de marche de plusieurs jours.

Les numéros d'appel téléphonique sont enregistrés sous forme de trains d'ondes à 400 Hz sur une des quatre pistes de la bande magnétique du magnétophone. Ils ne sont pas codés, mais chaque chiffre est représenté par une succession d'autant de trains d'ondes qu'il contient d'unités. Ce procédé économique de mise en mémoire des numéros d'appel est d'une utilisation aisée.

Les autres pistes portent des trains d'ondes à 1 000 et à 4 000 Hz, qui constituent soit des références pour le positionnement de la bande magnétique, soit des ordres pour la commande de la séquence d'appels.

Les réponses sont interprétées par un discriminateur unique se composant d'un amplificateur à contrôle automatique de gain suivi d'une bascule de Schmitt. La tension en créneaux qui en sort est appliquée aux deux fréquencemètres.

Le fréquencemètre principal est du type numérique à cinq digits pourvu d'une sortie binaire codée en BCD 1-2-4-8. La base de comptage utilisée pratiquement est de six secondes.

Un second fréquencemètre est du type enregistreur à plume et sa principale utilité est de permettre la vérification directe de la fiabilité des mesures effectuées. En effet, en cas de transmission défectueuse ou lors de la présence de parasites importants, la trace enregistrée sur le papier est un zig-zag plus ou moins large qui témoigne du mauvais fonctionnement d'une liaison ou d'un appareil appelé. De telles indications graphiques permettent de limiter les interventions d'entretien aux seuls appareils qui présentent des anomalies. Grâce aux deux fréquencemètres, la calibration des points de mesure est très aisée lors de l'installation sur le terrain, car le réglage définitif s'effectue par échange direct d'informations avec un opérateur resté au poste chef.

L'enregistrement sur bande perforée renseigne successivement le numéro d'appel téléphonique de la station, le ou les groupes de trois fréquences correspondant aux

mesures effectuées et à leur télé-étalonnage et, enfin, le groupe jour-heure-minute de l'instant de fin de communication. Ce dernier groupe de sept chiffres (trois pour le jour, deux pour les heures et deux pour les minutes) est fourni par un compteur électronique spécial, branché sur la roue des minutes de l'horloge. Comme il est essentiel de ne jamais perdre l'heure, le compteur est la seule partie de l'installation à être alimenté par une batterie d'accumulateurs qui assure une réserve de marche de quarante-huit heures.

Fonctionnement. En dehors de la commande de démarrage de la séquence d'appel par l'horloge, le poste chef est complètement contrôlé par des courants alternatifs de fréquences diverses provenant soit du magnétophone, soit de la station de mesure.

Après réception du signal habituel d'invitation à transmettre, l'envoi du numéro d'appel par le magnétophone provoque la perforation sur la bande de chaque chiffre à mesure qu'il est émis. Le courant de sonnerie alimente le décrocheur-raccrocheur qui "décroche" à la station de mesure et met l'oscillateur en communication avec le poste chef. La présence d'une réponse sur la ligne met les deux fréquence mètres en service ; l'interruption de cette réponse commande la perforation des résultats. Le code choisi pour la perforation des données brutes permet d'automatiser leur mise en tableau par un téléscripteur. L'examen de ce "listing" facilite la perforation de la bande corrigée destinée à l'ordinateur.

Plusieurs mesures peuvent se succéder en nombre quelconque, pourvu qu'elles soient séparées par des intervalles d'au moins une demi-seconde ; un silence de plus de deux secondes est interprété comme fin de transmission ; il provoque la libération de la ligne et la perforation du groupe jour-heure-minute.

Le poste chef passe immédiatement à l'appel suivant et ainsi de suite jusqu'au dernier. A ce moment, une cellule photo-électrique ordonne le rebobinage de la bande magnétique et l'arrêt du poste chef jusqu'au prochain signal de l'horloge.

Particularités de l'exploitation téléphonique. Comme on le sait, il n'est pas toujours possible d'obtenir rapidement une communication téléphonique. Aussi a-t-il fallu prévoir une suite logique d'opérations en cas de fonctionnement défectueux de la liaison.

Quand l'invitation à transmettre se fait attendre plus de trente secondes, le poste chef interrompt la ligne pendant deux secondes avant de reprendre l'écoute.

Au cas où le réseau est surchargé, l'avis de surcharge émis par le central téléphonique est reçu comme une réponse, mais ses caractéristiques permettent de déclencher immédiatement une procédure d'absence de réponse. Celle-ci consiste en la mise en mémoire du numéro appelé, qui sera réémis une ou plusieurs fois à la fin de la série d'appels. Si c'est la station de mesure qui ne répond pas, il est prévu une minute d'attente avant de commander la procédure d'absence de réponse.

Enfin, il arrive qu'une ligne dérangée émette un son continu qui serait interprété par le poste chef comme une réponse sans fin et qui le bloquerait indéfiniment. Ici encore, c'est la procédure d'absence de réponse qui se déclenche si la durée de la communication dépasse une minute.

CONCLUSION

Le système de télémesure LGR sur ligne téléphonique commutée convient à tout type de mesure à distance et particulièrement aux prélèvements d'informations sur les paramètres du milieu en sites naturels. L'ensemble réalisé permet la collecte d'un

grand nombre de mesures transmises sous forme de fréquences et automatiquement enregistrées de manière à garantir les valeurs relevées ainsi que leur traitement contrôlé et accéléré par calculateur ou ordinateur.

La simplicité, la robustesse, le faible encombrement et l'absence de toute source d'énergie autre que celle du réseau téléphonique public permettent une réduction appréciable de l'infrastructure des stations de mesure. Un réseau très fourni peut ainsi être constitué à parir d'un budget moyen, et le coût de la mesure est d'autant plus économique que le nombre de mesures à chaque station est plus élevé.

D'autre part, le poste chef peut être installé dans n'importe quel local pourvu d'une alimentation basse tension et d'un raccordement ordinaire au réseau téléphonique public, ce qui permet une large couverture territoriale.

Ce poste chef commande toutes les opérations de collecte et d'enregistrement de données de manière entièrement automatique. Il établit la séquence des appels aux stations et assure la perforation des résultats avec leur identification propre.

L'exploitation du système limite donc les interventions humaines, qui se bornent au contrôle centralisé du bon fonctionnement des stations du réseau complet et au transfert des données reçues aux ordinateurs.

Un tel ensemble peut être programmé de manière à fournir des signaux spéciaux en cas de dépassement de limites fixées, des tableaux ou des graphiques synoptiques pour l'étude de projets ou pour la recherche expérimentale.

Annexes

1. Fréquences et erreurs

L'onde porteuse étant exclue du circuit téléphonique normal, nous utilisons les fréquences vocales réglementaires.

La plus large plage continue de fréquence s'intercale en toute sécurité entre 600 et 1 800 Hz. En effet, la Régie des téléphones utilise en Belgique la fréquence de 450 Hz pour envoyer des signaux et elle coupe certaines fréquences nécessaires au système télex à partir de 1 940 Hz. Cette plage est très largement suffisante comme en témoigne l'exemple ci-après.

Un limnigraphe classique à flotteur présente couramment une erreur totale maximale de ± 1 cm pour une étendue de mesure de 2 mètres, soit 0,5 % de l'échelle.

Or, l'appareil LGR étant linéaire (annexe 2), l'erreur accidentelle de mesure peut être évaluée en périodes. Compte tenu d'une base de temps de 6 secondes pour le comptage, l'étendue de mesure comporte $(1\,800 - 600) \times 6 = 7\,200$ périodes. A l'erreur accidentelle de comptage qui atteint plus ou moins une période, il faut ajouter celle du télé-étalonnage valant aussi une période. L'erreur accidentelle maximale est dès lors de 0,028 % de l'étendue de mesure. Aussi peut-on, sans inconvénient, utiliser une fraction de l'échelle choisie : par exemple 1/10, voire 1/20.

2. L'oscillateur

La figure 1 montre le schéma du multivibrateur T_1, T_2 et de l'étage séparateur T_3 alimentés à tension constante à partir de la chute de tension aux bornes de la diode Zener ZM 10. Celle-ci est parcourue pendant la durée de la communication par le courant continu de ligne I_l. Le courant alternatif du signal de réponse, réinjecté dans la même ligne par le transformateur T_r, ne vaut que 1 à 4 % du courant continu.

Caractéristique linéaire. Dans le multivibrateur, quand un des deux transistors T_1 ou T_2 est bloqué, l'autre est saturé. Cela conduit à la courbe en créneaux $(V_{ce})_2$ de la figure 2, où il est supposé, à titre de simplification, que V_{be} et V_{ce} sont exactement nuls quand le transistor est saturé.

Négligeons les variations de V_{cc} pour montrer que la symétrie réalisée dans le multivibrateur permet le calcul de la fréquence en fonction de la demi-période. En effet (fig. 1 et 2) le

condensateur C_{21} a l'une de ses bornes au potentiel $(V_{bc})_1$ et l'autre à $(V_{ce})_2$. Au temps t_0, le potentiel $(V_{ce})_2$ tombe à zéro et $(V_{be})_1$ passe à $-V_{cc}$. A partir de cet instant, le courant entrant dans C_{21} à travers R_{b1} fait tendre $(V_{be})_1$ vers V_{bb} suivant l'expression :

$$(V_{be})_1 = -V_{cc} + (V_{cc} + V_{bb})\,[1 - \exp(-t/R_{b1}C_{21})] \qquad [1]$$

L'intervalle de temps t nécessaire pour que $(V_{be})_1$ devienne nul, définit la durée de la demi-période

$$t = \frac{T}{2} = \frac{1}{2f}$$

quand $(V_{be})_1$ est nul. On voit que la fréquence est liée au potentiel V_{bb} par une expression de la forme

$$e^{-1/y} = 1 - \frac{a}{a + x} \quad \text{pour} \quad x \geqslant a \qquad [2]$$

où $$a = V_{cc}\,,\ x = V_{bb} \quad \text{et} \quad y = 2RCf.$$

La figure 3 montre que cette expression est remarquablement linéaire dans la zone utilisée. En effet, la fréquence la plus basse s'établit au moment où $V_{bb} = V_{cc}$ $(x = a)$ et il y correspond $y_{min} = 1{,}442695$. L'autre point extrême est défini par la gamme de fréquences choisie (600 à 1 800 Hz) d'où $y_{max} = 3\,y_{min}$. L'abscisse correspondante est donc $x_{max} = 3{,}847323\,a$. Le plus grand écart par rapport à la droite qui joint ces deux points se produit environ au point ($x = 2{,}057638\,a$, $y = 1{,}75\,y_{min}$). Les coordonnées relatives de ce point sont donc

$$x \text{ relatif} = \frac{2{,}057638 - 1}{3{,}847323 - 1} = 0{,}371450$$

et

$$y \text{ relatif} = \frac{1{,}75 - 1}{3 - 1} = 0{,}375000$$

La déviation maximale par rapport à la droite n'est donc que de 0,355 % pour le multivibrateur seul. Cette courbure insignifiante est de plus compensée par deux propriétés non linéaires du circuit complet dues à la variation de V_{cc} et au capteur.

La figure 2 illustre l'évolution des potentiels aux deux bornes du condensateur C_{21}. A l'instant t_1, la différence de potentiel ne change pas instantanément : C_{21} se charge exponentiellement à travers R_{c2}, en un temps très court mais non nul. Ce courant de charge s'ajoute à celui qui traverse R_{c1} et T_1, ce qui provoque une chute fugitive de la tension V_{cc}. Il en résulte une variation concomitante de V_{bb} et une très légère distorsion par rapport à la figure 3.

Dans le capteur de la figure 1, le curseur est relié à un fil traversé par un faible courant qui lui aussi provoque une chute de tension.

En résumé, les écarts résiduels de linéarité sont si faibles qu'il n'est possible de les mettre en évidence qu'à grands frais de décimales. L'appareil est parfaitement linéaire.

3. Le décrocheur-raccrocheur (brevet déposé)

Le "décrocheur-raccrocheur" est basé sur les propriétés de circuits bistables dont la figure 4 est un exemple.

A l'état de repos, et pourvu qu'ils soient en silicium, les deux transistors T_1 et T_2 ne conduisent pas. Le condensateur C peut donc être chargé et conservé tel. Mais, au moment où la tension dépasse légèrement le seuil de conduction de la diode Zener Z, T_1 est rendu conducteur. Dès lors, T_1 et T_2 se maintiennent mutuellement à l'état saturé. Le courant initial suffit pour basculer le contact du relais bistable A et diminue ensuite à mesure que la tension aux bornes de C baisse.

Quand la tension et le courant sont devenus trop faibles pour assurer un tel fonctionnement, la conduction cesse subitement et l'impulsion positive ainsi créée au collecteur de T_1 est largement suffisante pour basculer l'étage suivant.

Un décrocheur-raccrocheur de ce type peut donc contenir plusieurs circuits semblables qui ont chacun une triple fonction : (a) préparer une mesure par l'intermédiaire d'un relais ; (b) attendre que le poste chef ait effectué cette mesure ; (c) donner l'ordre de passer au pas suivant.

La figure 5 est un exemple d'un décrocheur-raccrocheur utilisant ce principe pour réaliser le télé-étalonnage.

Le courant alternatif d'appel traverse le condensateur de blocage C_b et charge les condensateurs de stockage d'énergie C_1 à C_4.

Quand le seuil prévu est atteint, B_1 bascule et A_1 met l'oscillateur en service, le potentiel V_{bb} étant pris en 1, ce qui donne le point bas du télé-étalonnage. Ensuite A_2 commute la prise de potentiel V_{bb} en 2 pour la mesure proprement dite et A_3 la commute en 3 pour le point haut du télé-étalonnage. B_4 intervient en dernier lieu pour remettre les relais dans leur position initiale, ce qui coupe le courant continu d'alimentation et libère la ligne téléponique au central. L'appareil est ainsi prêt à recevoir un nouvel appel.

Le décrocheur-raccrocheur de la figure 6 procède différemment :

Le circuit bistable B_1 est alimenté à partir du courant d'appel et met l'oscillateur en service comme à la figure 5. L'énergie pour les autres fonctions est empruntée à la batterie centrale.

Chaque fois que C_2 est suffisamment chargé, ce qui demande plusieurs secondes à cause de la valeur élevée de R_2, B_2 bascule et excite le relais monostable B qui retombe bientôt à cause de la décharge de C_2. Le moteur M fait avancer d'un pas les contacts rotatifs m_1 et m_2, et s'arrête avec précision, freiné par m_3 ; m_1 prépare les mesures successives et m_2 provoque l'arrêt final après un tour complet, quand C_3 a eu le temps de se charger à travers R_3.

En dehors des courts instants de la commutation, ce dispositif ne consomme presque pas de courant continu, il ne peut donc pas perturber le fonctionnement de l'oscillateur. Il présente l'avantage de tous les dispositifs rotatifs, qui est de se prêter aisément à la multiplication du nombre de mesures effectuées successivement lors d'un même appel.

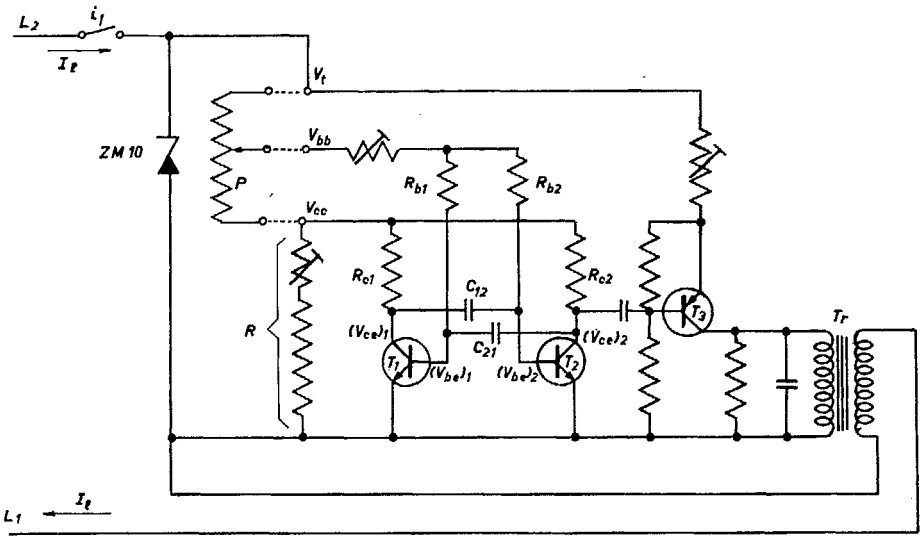


Figure 1 – Ensemble du circuit oscillateur (T_1, T_2) – capteur (P) relié à la ligne téléphonique (L_1, L_2) par l'interrupteur i_l et par le transformateur de sortie T_r

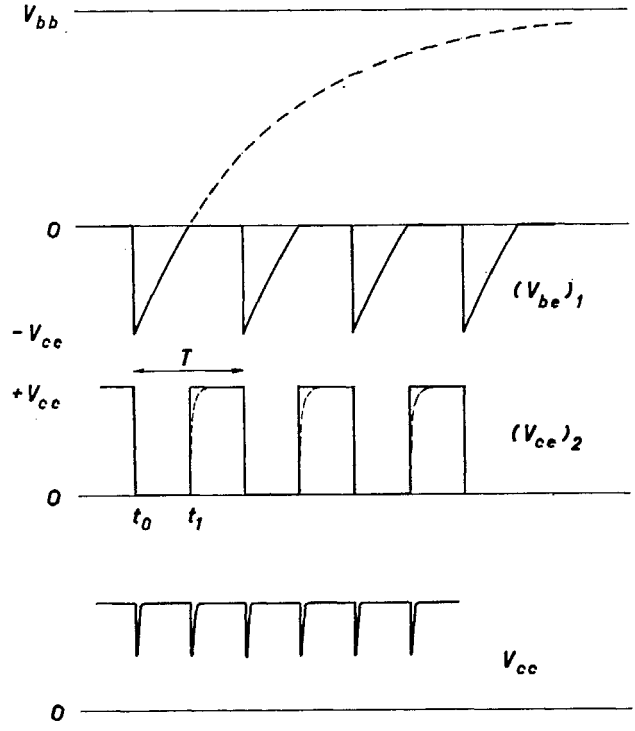


Figure 2 – Tensions aux bornes des transistors T_1 et T_2 du multivibrateur astable de la figure 1 en vue de la détermination de la période T

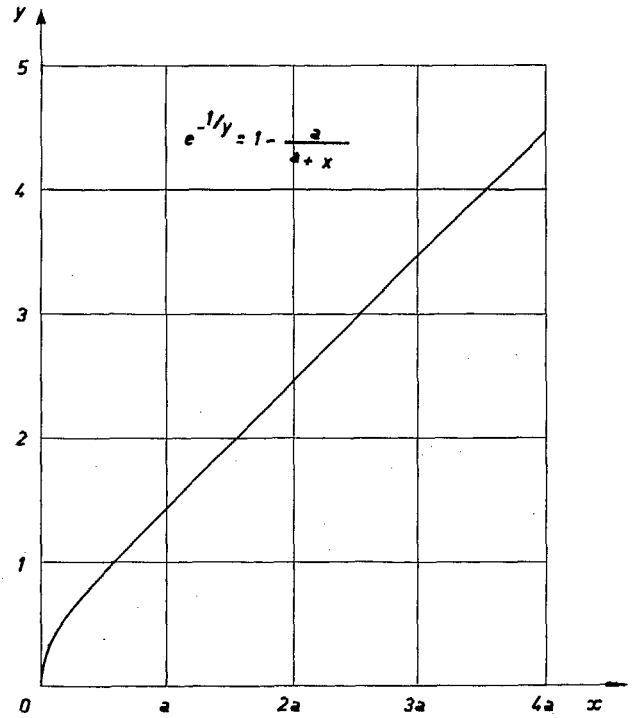


Figure 3 – Courbe caractéristique montrant la linéarité de la relation entre la fréquence de l'oscillateur ($y = 2RCf$) et la tension de commande ($x \approx V_{cb}$)' dans la zone de fonctionnement ($a \leqslant x \leqslant 4a$; $a = V_{cc}$)

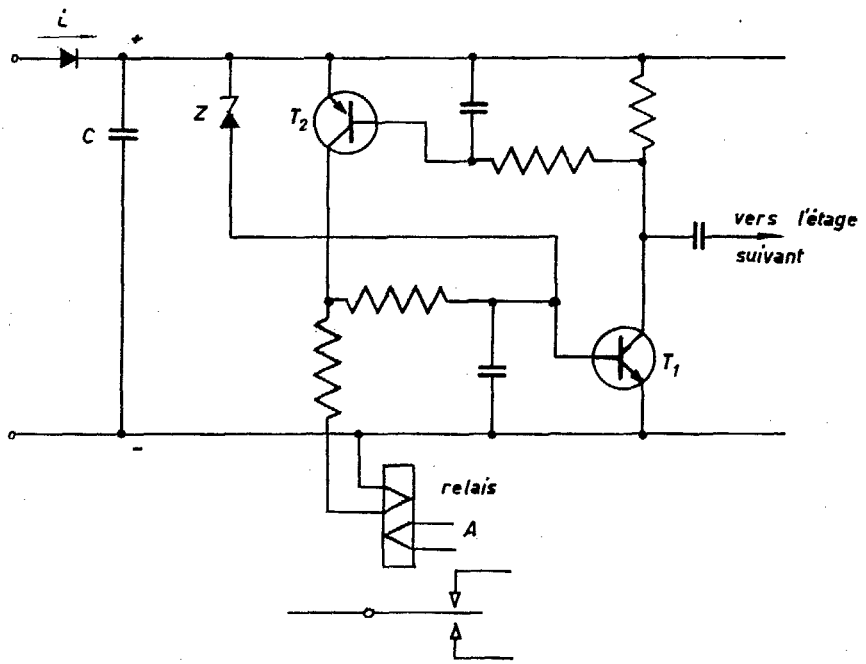


Figure 4 – Circuit bistable faisant partie du décrocheur-raccrocheur

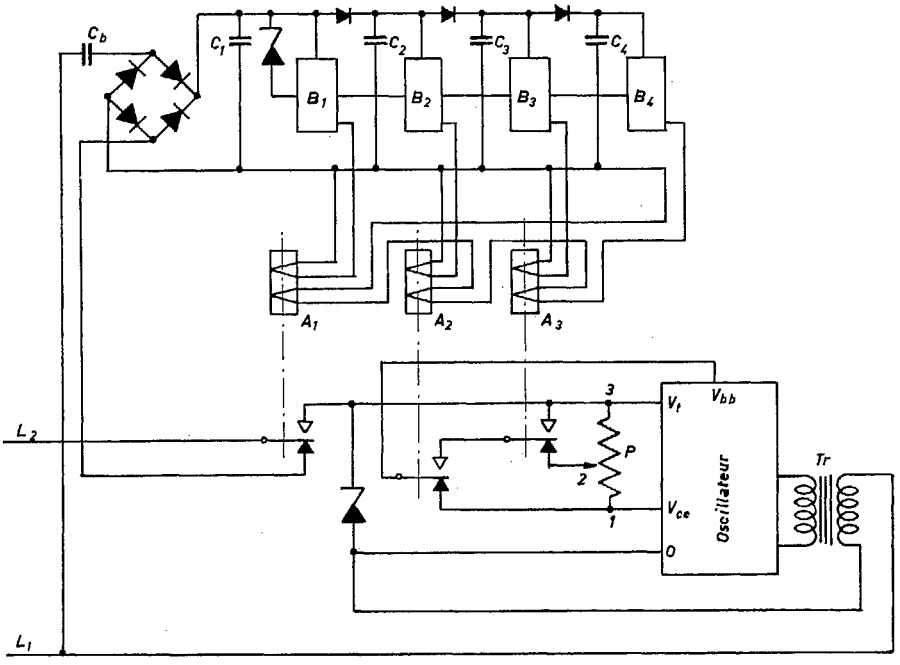


Figure 5 – Schéma complet d'une station de mesure dans lequel le décrocheur-raccrocheur composé de quatre circuits bistables (B_1 à B_4) réalise le télé-étalonnage au niveau d'un point de mesure

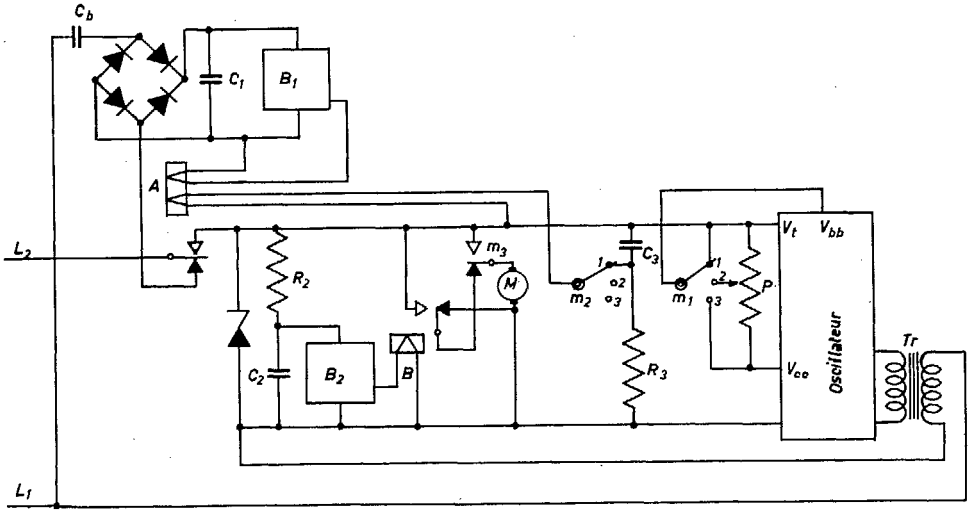


Figure 6 – Schéma complet d'une station de mesure avec décrocheur-raccrocheur pouvant réaliser le télé-étalonnage d'un grand nombre de points de mesure grâce à des contacts rotatifs actionnés par le moteur *M*

Nouveaux limnigraphes et automatisation de la gestion d'un réseau hydrométrique en France

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Résumé. Dans une première partie, les auteurs décrivent les principaux maillons constituant la chaîne de gestion d'un réseau hydrométrique d'abord tels qu'ils sont, ensuite tels qu'ils sont amenés à évoluer devant l'introduction des systèmes modernes de collecte et de traitement des données. La nécessité d'une profonde mutation des équipements de mesure a conduit les auteurs à lancer la mise au point de plusieurs prototypes de nouveaux limnigraphes, qu'ils décrivent ici en rendant compte des expérimentations et essais.

En conclusion, les auteurs présentent quelques exemples de chaîne de gestion moderne pour un réseau hydrométrique qui ont été retenus par divers organismes de France.

NEW LIMNIGRAPHS AND THE AUTOMATION OF THE MANAGEMENT OF A HYDROMETRICAL NETWORK IN FRANCE

Abstract. The authors describe the principal links of the management chain of a hydrometrical network, both as they are at present and as they have evolved due to the introduction of modern data acquisition and processing systems. The need for many transformations in measuring equipment has led the authors to start working on several prototype limnigraphs, which are described. The results of the tests which have been carried out on them are given.

In conclusion, the authors present some examples of modern management techniques of a hydrometrical network which have been adopted by various organizations in France.

NUEVOS LIMNÍGRAFOS Y AUTOMATIZACIÓN DE LA UTILIZACIÓN DE UNA RED HIDROMÉTRICA EN FRANCIA

Resumen. El conocimiento de la red de medidas y la necesidad de una política de utilización de la red, a las que se refiere la introducción, han llevado a varios responsables de redes hidrométricas de Francia a investigar una organización óptima.

En una primera parte, los autores describen los principales eslabones que constituyen la cadena de gestión de una red hidrométrica, en principio tal como son, y a continuación tal como se ven obligados a evolucionar ante la introducción de los sistemas modernos de toma y tratamiento de los datos. La necesidad de una profunda mutación de los equipos de medidas ha conducido a los autores a lanzar la puesta a punto de varios prototipos de nuevos limnígrafos que describen,y se explican los experimentos y ensayos realizados.

Como conclusión, los autores presentan algunos ejemplos de explotación moderna de una red hidrométrica de diversos organismos de Francia (Ministerio de Agricultura, Oficina de Investigación Científica y Técnica de Ultramar).

НОВЫЕ ЛИМНИГРАФЫ И АВТОМАТИЗАЦИЯ УПРАВЛЕНИЯ ГИДРОМЕТРИЧЕСКОЙ СЕТЬЮ ВО ФРАНЦИИ

Резюме. Знание измерительной сети и необходимость управления ею, о которой упоминается во введении, заставили некоторых гидрологов, имеющих дело с гидрометрическими сетями во Франции, искать оптимальную систему для управления ими.

В первой части авторы описывает основные звенья в технике управления гидрометрической сетью, это прежде всего то, что они представляют собой сейчас, и далее — как они развились в связи с введением современных систем получения и обработки данных. Необходимость многих преобразований в измерительном оборудовании заставила авторов начать работу над несколькими образцами новых лимниграфов, описание которых дано наряду с результатами проведенных экспериментов и испытаний.

В заключение авторы приводят некоторые примеры техники современного управления гидрометрической сетью, которая была примята различными организациями во Франции (Министерство сельского хозайства, ORSTOM).

POSITION DU PROBLEME

Généralités

Le développement de plus en plus rapide du nombre de stations hydrométriques installées par le Ministère de l'agriculture, dans le cadre général des attributions du Service de l'hydraulique en matière de police des eaux, a conduit le chef de ce service à s'assurer le concours du Service hydrologique de l'ORSTOM pour l'étude de l'extension rationnelle du réseau hydrométrique du Ministère de l'agriculture ; cette étude a été conduite en liaison avec la Section hydrologique du CERAFER.

Cette étude a, par ailleurs, été coordonnée, d'une part, avec celles promues par la Délégation générale à la recherche scientifique et technique et, d'autre part, avec celles menées par d'autres services gestionnaires de réseaux en France, notamment la Division technique générale de l'Electricité de France.

Situations présente et à venir du réseau hydrométrique du Service de l'hydraulique

L'enquête menée en 1967 mettait en relief les caractères suivants.

Les stations ressortissant de la compétence du Ministère de l'agriculture représentaient un tiers du total des stations existantes (de l'ordre de 1 650 à l'époque) en France ;

Ces stations étaient récentes (90 % d'entre elles ayant moins de dix ans).

Le nombre de ces stations tendrait à croître rapidement, face aux autres secteurs, eux-mêmes en faible expansion ou stables (les besoins de la France pouvaient être estimés à quelque 900 nouvelles stations dans les dix années à venir, dont les trois quarts seraient de la compétence du Ministère de l'agriculture).

L'équipement limnigraphique était important (85 %) et homogène (90 % de limnigraphes à flotteurs OTT).

Le traitement des données recueillies (essentiellement limnigrammes) était à peu près exclusivement manuel.

Si l'on considère que la situation actuelle reste qualitativement semblable, il faut préciser cependant que le nombre de stations installées a crû très rapidement et dépasse très probablement le total de 800. Dans de telles conditions les solutions de gestion retenues peuvent se classer en deux groupes : les solutions transitoires destinées à permettre la modernisation du traitement de l'information recueillie tout en conservant l'équipement limnigraphique existant, non amorti, soit tel quel, soit légèrement modifié ; les solutions d'avenir destinées à l'équipement des stations futures et faisant appel aux technologies les plus évoluées.

Les solutions du premier groupe sont bien entendu des compromis, mais certaines d'entre elles peuvent constituer, nous le verrons, d'excellentes solutions d'avenir, du moins à moyen terme et en tout cas dans des conditions économiques remarquables.

Situation particulière des réseaux hydrométriques gérés par l'ORSTOM

L'ORSTOM assure directement ou par le truchement d'une convention la gestion du réseau hydrométrique dans 4 départements et territoires d'outre-mer de la France et dans 11 Etats africains francophones sans compter la collaboration qu'il apporte aux services gestionnaires d'autres Etats tels que la Tunisie et l'Algérie.

L'ensemble des 15 réseaux gérés par l'ORSTOM présentait les caractères suivants en 1969 : (a) plus de 600 stations, dont 110 seulement équipées de limnigraphes (90 % étant de marque OTT) ; (b) une croissance d'ici à 1975 estimée à 70 stations nouvelles et 100 nouveaux limnigraphes ; (c) un traitement entièrement manuel des limnigrammes et relevés des échelles limnimétriques.

Dans le proche avenir, l'ORSTOM orientera sa gestion de réseau vers une automatisation du traitement des données, en conservant provisoirement l'équipement classique de mesures, orientation imposée par les conditions locales (faible habitat, inaccessibilité) et par la nécessité d'un traitement au niveau de l'Etat avant la centralisation à Paris.

La mesure du niveau, élément d'une chaîne de gestion

La recherche de conceptions nouvelles dans le domaine du limnigraphe apparaît donc comme une nécessité devant l'accroissement considérable du parc de limnigraphes et l'inéluctable obligation de passer de la gestion manuelle, pratiquement seule possible avec les limnigraphes actuels, si satisfaisants soient-ils par ailleurs, à la gestion automatique.

Il ne s'agit pas alors de rechercher la solution parfaite du seul problème de la mesure et de l'enregistrement des niveaux et il est indispensable de tenir compte de l'étendue complète de la "chaîne de gestion" du réseau hydrométrique, qui comprend : (a) le captage de l'information "hauteur d'eau" ; (b) la collecte et la transmission de cette information du lieu de captage au centre de gestion ; (c) la transformation de l'information "hauteur d'eau" en "information débit" ; (d) le traitement de l'information "débit" : élaboration des données de base ; classement et archivage ; analyses scientifiques, etc.

Le tableau 1 détaille quelque peu l'énumération ci-dessus, et nous ajouterons les remarques suivantes quant à la collecte et à la transmission de l'information "hauteur" :

Ces opérations peuvent être réalisées par la seule intervention de l'homme (y compris bien entendu les moyens classiques de transport, notamment l'automobile et le courrier postal) : cette solution a seule été retenue pour le secteur de l'espace rural (Ministère de l'agriculture) et pour les réseaux ORSTOM.

Elles peuvent être réalisées simultanément par la télétransmission : cette solution est intéressante, d'une part, pour les régions à faible densité d'habitat et, d'autre part, pour les réseaux d'alerte (inondations notamment) ou de contrôle (usines hydro-électriques, canaux de navigation, etc.).

LES EQUIPEMENTS LIMNIGRAPHIQUES

Caractéristiques optimales du capteur de niveau

Si l'on considère le problème en soi du captage de l'information "hauteur d'eau" et de son codage, il est possible de distinguer trois plans.

Tableau 1. *La chaîne de gestion d'un réseau hydrométrique.*

Opération de gestion	Réalisation pratique	Equipement	Forme de l'information	
1 Captage, codage de l'information. Hauteur H	Observations des hauteurs d'eau fonction du temps (H , *t*)	Limnigraphe	Limnigramme Bande/ruban	Secteur hydrométrique — Terrain
2 Captage de l'information Etalonnage E	Mesures de débits	De jaugeages	Carnet de jaugeages	
3 Codage de l'information Etalonnage E	Dépouillements des jaugeages Tracé courbes de tarage	De bureau	Liste de jaugeages Courbe de tarage	Secteur hydrométrique — Bureau
4 Critique et contrôle des informations H et E	Corrections erreurs, lacunes, etc.	De bureau	–	
5 Transcodage de l'information H	1 Dépouillement limnigrammes	Analyseur de courbe	Chroniques de coordonnées (X , Y)	
	2 Mise chroniques (H , *t*) sur support compatible (s.c)	Transcodeur Programmes LIMNI sur ordinateur	Chroniques (H , *t*)*	
6 Transcodage de l'information E	Mise en équation des courbes de tarage, sur s.c.	Programmes TARAG sur ordinateur	Caractéristiques des fonctions de tarage*	
7 Traitement de l'information H	1 Transformation hauteur H – débit Q	Programmes TRAD sur ordinateur	Chroniques (Q , *t*)	Centre de calcul
	2 Elaboration données de base "Hauteurs"	Programmes HAUT sur ordinateur	Données de base "Hauteurs"*	
8 Traitement de l'information Débit Q	1 Elaboration données de base "débits"	Programmes DEBIT et programmes ANNUA sur ordinateur	Données de base "Débits"	
	2 Publication Annuaire		Annuaire	
9 Traitements scientifiques sur débits Hauteurs	Etudes spéciales d'interprétation et données pour aménagements.	Programmes spéciaux sur ordinateur	Rapports imprimés	

*Toutes ces informations sortent sur support compatible et sur tableaux d'imprimante.

Caractéristiques inhérentes à la nature du réseau. L'importance relative des stations qualifiées de "tertiaires", selon l'usage maintenant consacré, confère au réseau étudié un caractère certain de mobilité, qui exige : coût minimal de l'infrastructure ; récupération maximale de l'équipement en vue des déplacements ; universalité de l'appareillage (cours d'eau et écoulements de types très variés).

La faible qualification du personnel de collecte de l'information exige : installation facile ; maniements aisés ; robustesse.

Caractéristiques propres au capteur. Nous distinguerons les aspects suivants :
Bande de mesure. En général 0-10 m, avec possibilité d'extension à 20 m.

Précision et sensibilité. Celle-ci pourrait être variable, selon les normes ci-après : ± 1 cm entre 0 et 2 m ; ± 2 cm de 2 à 10 m ; ± 3 cm au-delà de 10 m.

Cadence d'enregistrement. Le pas de temps fixe doit pouvoir être modifiable à volonté entre les extrêmes 1/4 d'heure et 4 heures (intermédiaires = 1/2 h – 1 h – 2 h) ; une cadence au 1/10 d'heure est souhaitée (petits cours d'eau).

Autonomie. Le principe retenu est celui de la source d'énergie autonome et il est souhaitable que la durée d'autonomie soit de 1 à 3 mois, et éventuellement 6 mois.

Caractéristiques propres au codeur. Trois idées directrices ont été dégagées : (a) fidélité sévère du codeur ; dans le cas du codage discontinu non clair : (b) simplicité du convertisseur, ou transcodeur, pour la phase de production du support compatible avec l'ordinateur ; (c) nécessité d'un contrôle visuel, sous la forme d'un enregistrement (type limnigramme) constituant un témoin en modèle réduit (les caractéristiques de ce témoin devraient être de cet ordre : 100 mm de largeur ; 1/20° à 1/50° d'échelle de réduction ; 0,5 à 2 mm/h de vitesse de déroulement).

Inventaire des dispositifs de captage et de codage: situation du marché

A priori il n'existe aucune incompatibilité entre les dispositifs de captage et ceux de codage, de telle sorte que toutes les combinaisons entre un type de limnigraphe et un dispositif de codage quelconque sont concevables et que chacune de ces combinaisons présente des avantages, mais aussi des inconvénients.

Un inventaire général a été entrepris dans cet esprit et nous ne pouvons que renvoyer pour tous détails à la publication correspondante [1][1]. Signalons que cet inventaire n'était pas exhaustif : d'une part étaient écartées les solutions non réalisables sur le terrain ou d'un prix de revient inacceptable en réseau, pour ne retenir que les solutions existantes, éventuellement modifiées ou devant l'être, et les solutions à un stade avancé des études ou des réalisations ; d'autre part l'attention s'est portée essentiellement sur le marché européen, les références au marché américain n'étant faites que par souci de comparaison.

Cet inventaire permettait notamment de dresser un tableau du marché, que nous avons complété et mis à jour pour constituer le tableau 2.

Au total, et par comparaison avec les caractéristiques optimales définies précédemment, il est possible d'éliminer un certain nombre de combinaisons pour n'en retenir que six, qui peuvent alors se classer ainsi.

Limnigraphes à flotteur, à sortie sur BP et témoin visuel. Le prototype des établissements Richard n'a toujours pas été commercialisé ; par contre la société OTT a commercialisé largement le prototype dont les essais, réalisés sur les bassins versants représentatifs de l'Orgeval (gérés par le CERAFER), ont été présentés en mars 1969 à la Société hydrotechnique de France (SHF) [2].

Adjonction à un limnigraphe classique d'un codeur magnétique. Cette solution, élégante et particulièrement apte à la valorisation d'un équipement existant, a été exposée en mars 1969 à la SHF [3] ; la réalisation est due à une collaboration entre les ingénieurs français de l'EDF-DTG et italiens de l'ENEL, l'idée étant de P. Guillot.

Limnigraphes à insufflation de gaz. Deux prototypes existent, qui seront décrits ci-après ; l'un a été financé par la Délégation générale à la recherche scientifique et technique, l'autre par le Ministère de l'agriculture.

Limnigraphes à prise de pression par jauges de contrainte. Deux prototypes existent, décrits ci-après, et financés par les deux organismes cités ci-dessus.

1. Les chiffres entre crochets renvoient à la bibliographie en fin d'article.

Tableau 2. *Information limnimétrique. Situation du marché.*

Principes (capteur) + (codeur)[a]		Appareillages correspondants		
		Fabrications de série	Prototypes[b]	A l'étude
2.1. Limnigraphes à flotteur	2.11 L	OTT ; Richard ; Stevens		
	2.12 BP	OTT ; Stevens + F & P		
	2.13 BP + T	OTT	*Richard*	
	2.14 BM pré-enregistrée	Limniphone SAREG ; OTT		
	2.15 Imp. électr.	Stevens Telemark		
	2.16 L + BM		*DTG*	
2.2. Limnigraphes à insufflation de gaz	2.21 L	Telimnip Neyrpic ; Balance Rittmeyer		
	2.22 BP	Stevens (ou CAE) + F & P		OTT
	2.23 BP + T			
	2.24 RN + T		*Neyrpic*	
	2.25 BM		*CDC*	
2.3. Limnigraphes à prise de pression pneumatique	2.31 L	Hydrocapteur Richard		
2.3. Limnigraphes à prise de pression par jauge de contrainte	2.32 RN		*SFIM*	
	2.33 BM		*CDC*	

a. L = Limnigrammes ; T = Témoin ; BP = Bande perforée ; BM = Bande magnétique ; Imp. électr. = Impulsion électrique ; RN = Ruban numérique.

b. Les prototypes disponibles pour essai sont en italique.

Etat d'avancement des prototypes cités ci-dessus.

Le limnigraphe à insufflation de gaz des établissements Neyrpic

La description détaillée a été faite dans un texte antérieur [2] et nous ne donnerons ici qu'un résumé, sans pouvoir reproduire les documents photographiques inclus dans la publication citée.

Le capteur est, à quelques détails près, celui qui constitue l'essentiel du limnigraphe commercialisé sous le nom de Telimnip : une bouteille d'air comprimé, dotée d'un manodétendeur, est reliée par des tuyauteries à une prise de pression d'air, d'une part, et à un manomètre à mercure, d'autre part.

La nouveauté se présente comme suit : (a) la mesure du niveau de mercure par une sonde palpeuse à deux électrodes (ce dispositif fait cependant partie des fabrications de série de la société), le contact permanent étant assuré par un moteur commandé, en montée ou en descente, par un circuit à transistors ; (b) l'addition d'un compteur indicateur (contrôle des lectures) relié mécaniquement à un compteur imprimant, produisant donc un ruban numérique (caractères compatibles avec un lecteur optique IBM 1285 ou 1287) ; (c) l'addition d'un enregistreur témoin électromécanique classique.

Ce prototype, financé par le Service de l'hydraulique du Ministère de l'agriculture, a été essayé sur le terrain (Service régional d'aménagement des eaux Languedoc-Roussillon) d'avril 1969 à janvier 1970. Cet équipement est en cours de modification, sinon dans son principe du moins pour certaines parties, et l'expérience acquise est

telle que les modifications porteront peut-être sur la sonde palpeuse à deux électrodes (fiabilité en milieu très humide) et sans doute sur le compteur imprimant (les difficultés rencontrées sur ce point ont toutes été levées successivement, mais le problème reste celui du prix de revient). De nouveaux résultats seront certainement acquis en septembre 1970.

Enfin, cette expérience a été particulièrement intéressante en mettant en évidence les difficultés de production, dans les conditions de terrain, d'un ruban numérique compatible avec un lecteur optique.

Le limnigraphe à prise de pression par jauge de contrainte de la SFIM

Le prototype a été également financé par le Ministère de l'agriculture et la description faite de même dans le texte sus-visé [2]. Nous ne reprendrons pas ici la description détaillée, afin de pouvoir insister sur l'expérience acquise.

Nos espoirs et nos efforts s'étaient concentrés sur la partie "capteur", considérée comme originale, sinon dans son principe du moins dans son application à l'hydrométrie. Les essais faits au laboratoire d'hydraulique du CERAFER ont montré que les performances annoncées étaient réalisées en linéarité (meilleure que 0,3 %), en hystérésis (meilleure que 0,1 %) et en fidélité (0,1 %).

Si l'on écarte les défauts légers constatés sur l'isolation de la jauge de contrainte elle-même (type DYNISCO-GW 22), l'ensemble capteur-convertisseur a donné toute satisfaction, et notamment l'électronique incorporée au boîtier étanche.

Une autre source de satisfaction est que l'ensemble de mesures continue de fournir des données cohérentes, et sans dérive, lorsque la batterie se décharge ; l'arrêt est brutal lorsque la batterie est trop déchargée, et seules les deux ou trois dernières lectures (pas de temps = 1/4 d'heure) sont douteuses.

C'est en cours de fonctionnement avec le secteur que les plus grosses difficultés ont été rencontrées, notamment lors de brusques changements de tension et de coupures ; des perturbations interviennent alors au niveau de l'imprimante Olympia (impressions ininterrompues ; impressions parasites et totalement aléatoires) ; elles n'ont pas pu être totalement annulées. Par ailleurs, la consommation de l'ensemble s'avère très élevée, comme prévu d'ailleurs [2] ; l'autonomie n'est guère que de l'ordre de 48 heures.

Nous rappellerons que, dans ce prototype, c'était la partie "capteur" qui nous intéressait le plus et les essais en laboratoire ont été assez satisfaisants pour que l'équipement soit installé sur le terrain (bassins représentatifs de l'Orgeval) au printemps 1970, afin de poursuivre les essais du capteur-convertisseur et d'examiner de plus près les problèmes de lecture du ruban numérique (notamment celui des mentions portées manuellement).

Les prototypes mis au point par la Compagnie des compteurs

La Délégation générale à la recherche scientifique et technique, soucieuse, dans le cadre des problèmes d'estimation des paramètres d'environnement, de favoriser l'amélioration de l'appareillage de mesure et des systèmes de dépouillement, a confié à la Compagnie des compteurs la réalisation de stations polymesures (à 15 directions au maximum), dont une version simplifiée, dite "trimesure", est présentée ci-après.

La station "trimesure" (en fait une quatrième voie est disponible) est plus spécialement adaptée au contrôle des ressources, les grandeurs mesurées et codées étant en effet : niveau d'eau (et plus généralement toute pression) ; hauteur des précipitations ; température de l'eau. Nous ne parlerons ci-après que de la voie "niveau d'eau".

Mesure du niveau par manomètre à jauges de contrainte. Nous n'insisterons pas sur ce prototype dont le principe n'est pas très différent de celui réalisé par la SFIM (voir ci-dessus), et nous dirons simplement qu'il est caractérisé par : (a) le fait que les capteurs sont fabriqués par les Etablissements associés Jules Richard ; (b) une électronique originale, à très haute stabilité et très grande précision (mesures à 0,2 % près) de dimensions réduites et de consommation électrique modérée (40 mA sous 24 V).

Mesure du niveau par manométrie à ruban vibrant. Ce prototype correspond à une application particulièrement originale, et délicate, d'un principe connu.

Principe (voir fig. 1*a*). Un ruban d'acier magnétique B est tendu entre un point fixe réglable C et un soufflet S_1 sur lequel est appliquée la pression p_1 correspondant à la hauteur d'eau à mesurer. Il peut vibrer selon le plan de figure, dans le champ H, perpendiculaire au plan de figure, produit par deux aimants permanents. Ce mouvement induit dans le ruban une tension alternative dont la fréquence est fonction de la traction exercée sur lui, par le système S_1 et ressort de faux zéro D. Ce signal est amplifié dans le circuit électronique F alimenté par une source + et − 9 volts, le signal de sortie J étant appliqué à une bobine d'excitation E qui actionne le ruban sur le mode fondamental. L'information J est délivrée sous la forme d'une tension alternative d'amplitude constante (8 volts de crête à crête), dont la fréquence f est reliée à la pression p_1 à mesurer par la loi théorique :

$$f = \mathrm{M}\ \sqrt{p_1 . \mathrm{S}_1 + \mathrm{D}}$$

dans laquelle M est une constante dépendant de la longueur et de la masse du ruban, et D représente la traction initiale du ressort de faux zéro. En fait la loi pratique est légèrement différente (variation de la longueur du ruban en fonction de la traction subie) et il s'ensuit une erreur absolue systématique dont on peut tenir compte une fois pour toutes.

Versions constructives. On peut adopter deux montages différents : (a) le manomètre sert à mesurer la pression dans un circuit d'insufflation d'air par buse immergée ; c'est le cas de la figure 1.*a*; (b) le manomètre peut être lui-même immergé, la mise en équilibre atmosphérique étant assurée par un tube remontant à la surface avec le câble électrique d'alimentation et de sortie du signal (fig. 1.*b*). Par ailleurs, on peut disposer de plusieurs calibres de mesure (0,04 à 1,2 m ; 0,20 à 7 m ; 0,80 à 20 m), chacun d'eux étant capable d'une fourchette de 1 à 35 environ.

Performances. Nous n'insisterons que sur les propriétés les plus remarquables, à savoir :

Résorption partielle de l'hystérésis en cas de perturbations relativement lentes, ce qui est le cas, même pour les crues les plus brutales (la vibration du ruban facilite la reprise des positions relatives des différentes pièces).

Tolérance de ± 40 % sur la variation de la tension d'alimentation.

Forme parabolique de la loi de réponse du manomètre conférant à l'appareil une sensibilité meilleure dans le bas de l'échelle qu'en haut (pour 1 cm d'eau la variation de fréquence passe respectivement de 2,9 Hz à 1,1 Hz environ.

Exploitation des informations. L'un comme l'autre de ces manomètres devant normalement être utilisés en réseau, il suffit que la station locale correspondante délivre un signal numérique, "stocké" sur place ou télétransmis, la traduction en hauteurs d'eau pouvant se faire au poste central de traitement.

Pour le premier appareil, la fréquence est mesurée par un simple compteur, associé à la base de temps de la station. Pour le second, la tension analogique est convertie en numérique.

Le mode d'enregistrement local qui a paru le plus approprié aux conditions d'un service autonome est la bande magnétique. Cependant, les stations expérimentales sont pourvues, en outre, d'une imprimante qui enregistre les hauteurs en centimètres, pour confrontation directe avec une échelle témoin.

Les deux versions de ce prototype sont actuellement aux essais dans les laboratoires de la CDC, avec le prototype à jauge de contrainte. Il a été envisagé de les expérimenter avant la fin de 1970 d'abord sur une écluse de la Seine (fort marnage de 4 m plusieurs fois par jour), puis sur les bassins de l'Orgeval.

LES CHAINES DE GESTION

Nous avons déjà dit que les combinaisons entre un type donné de limnigraphe et un dispositif de codage quelconque étaient toutes concevables. Si l'on ajoute qu'il est toujours possible de passer d'un type de support à un autre par une ou plusieurs opérations intermédiaires, le nombre de combinaisons de chaînes de gestion concevables est alors pratiquement illimité et chacune d'elles présente des avantages, mais aussi des inconvénients.

L'ensemble des problèmes qui se posent pour le traitement automatique (nature des supports ; transcodage ; nature des accès et compatibilité avec le type de traitement, scientifique ou de pure gestion ; vitesses d'entrée et de sortie ; capacité des supports et concentration de l'information ; fiabilité et longévité des supports) a fait l'objet d'un exposé très complet [1] qui ne saurait être résumé ici.

Par contre, nous pouvons présenter, sous la forme condensée du tableau 3, les combinaisons d'équipement possibles qui fournissent les meilleurs réponses aux multiples problèmes que pose la mise en place d'une chaîne de gestion rationnelle de réseau hydrométrique.

Dans le cas présent notre choix est guidé par le souci que nous avons d'améliorer les conditions d'exploitation du réseau hydrométrique, en valorisant au mieux un équipement qui est encore très loin d'être amorti et, dans ces conditions, il est possible de distinguer deux grands types de combinaisons, compte tenu de la préexistence d'un réseau hydrométrique non amorti et de son extension future probable.

Les chaînes de gestion incluant les limnigraphes actuellement en service

On peut distinguer encore les trois types de solutions ci-dessous.

L'équipement liminigraphique existant reste inchangé. La seule solution est alors celle de l'analyseur de courbe et l'on peut avoir deux cas différents:

Pour un réseau hydrométrique très homogène en matériel et au format stabilisé, on peut envisager d'utiliser un analyseur de courbe automatique, ou même réaliser un appareil spécifique adapté aux seuls besoins de ce réseau, et donc d'un prix de revient limité [1, 3].

Pour un réseau hydrométrique moins homogène et en pleine expansion, il est prudent d'acquérir un appareil à performances aussi universelles que possible (largeurs et échelles des limnigraphes ; retournements du stylet ; pointé en pas de temps libre ou préréglé selon différents intervalles).

Pour le réseau hydrométrique du Ministère de l'agriculture et pour ceux du Service hydrologique de l'ORSTOM, c'est l'analyseur semi-automatique qui a été retenu, et la prospection du marché a permis de retenir :

Pour le Ministère de l'agriculture : le lecteur de courbes D.MAC Pencil Follower (type P.F. 1000 – Mark 1B) ; cet appareil offre de larges possibilités de lecture (y

Tableau 3. *Présentation des combinaisons d'équipement possibles.*

Plan opérationnel	Equipement futur				
	Stations existantes				
	1	2	3	4	5
Limnigraphe	OTT X, XX, R.16, Télimnip	(OTT X, XX, R.16, Télimnip) + Codeur BM (DTG)	OTT type 20.061 à témoin	Prototypes Neyrpic (insufflation + sonde à 2 électrodes) et SFIM (capteur à jauge de contrainte)	Prototypes CDC *a*) Insufflation et ruban vibrant *b*) Capteur à jauge de contrainte
Information hauteur captée	Limnigramme	Limnigramme + BM	BP_c (code Télex)	RN_c	BM
Transcodage	Analyseur de courbe	Lecteur-enregistreur magnétique	Pas d'équipement	Pas d'équipement	Lecteur-enregistreur magnétique
Information hauteur à traiter	CP(ou BP_c ?)	BM_c	BP_c	RN_c	BM_c
Entrée ordinateur	Lecteur CP	Lecteur BM	Lecteur BP	Lecteur optique	Lecteur BM
Programmes	CP	CP	CP	CP	CP
Sortie débit	Imprimante	Imprimante	Imprimante	Imprimante	Imprimante
Classement information					
Tr. de gestion	CP/BP/BM	BM	(CP)/BP/BM	RN/(CP)/BM	BM
Tr. mixte	CP/(DM ?)	(DM ?)	CP/(DM ?)	RN/CP/(DM ?)	(DM ?)

N.B. Les solutions entre parenthèses sont relatives à des supports possibles mais non sûrs, ou non logiques dans le cas traité.
L'indice *c* accolé à un support CP, BP, signifie que celui-ci est compatible avec un ordinateur.
BM = Bande magnétique ; CP = Carte perforée ; DM = Disque magnétique.

compris celle du travail sur micro-film) et de sortie (imprimante ; carte perforée ; bande perforée ; bande magnétique) ; la sortie sur perforatrice de cartes a été retenue au départ (traitement scientifique à prévoir) et deux équipements seront mis en place dans le courant de l'été 1970, avant toute généralisation.

Pour l'ORSTOM : le lecteur de points BENSON-LP 620, d'un prix plus réduit. Cet appareil a une sortie unique sur perforatrice de cartes, cette solution étant retenue en raison, d'une part, de l'importance du traitement scientifique et, d'autre part, de l'existence de données limnimétriques sous forme de lectures d'échelles (encore 80 % de l'ensemble de l'information), faisant en tout état de cause l'objet d'une perforation de carte pour traitement. Le support carte perforée s'impose donc.

L'équipement limnigraphique existant est modifié. Cette modification peut être:

Importante : en général on procède par adjonction d'un second limnigraphe, codeur celui-là ; cette solution est coûteuse et nous ne l'admettons que lorsqu'elle apporte en même temps une amélioration de la fiabilité (par exemple dans les régions difficilement accessibles, ou aux climats très froids, tel le Canada).

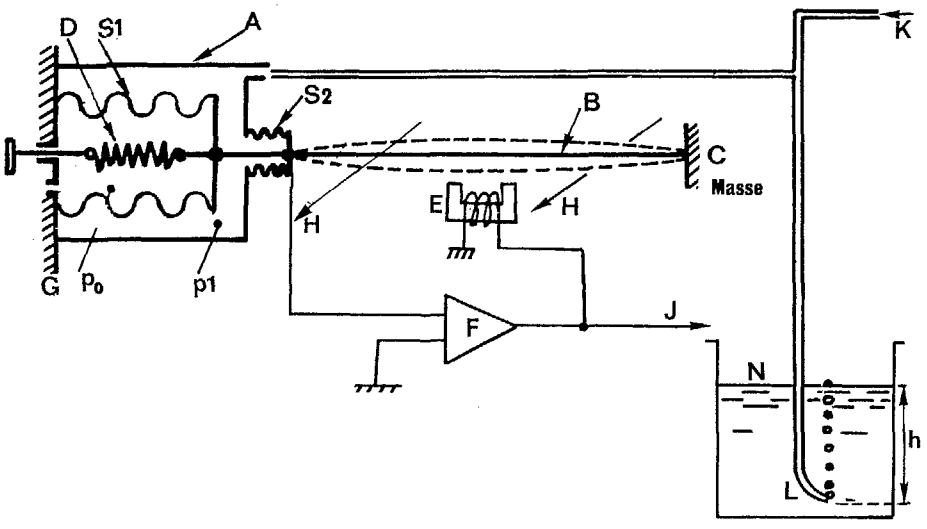
Limitée : le codeur magnétique, préconisé par la Division technique générale de l'EDF (voir ci-dessus, p. 696), nous paraît être une solution à la fois élégante et économique. Cette solution est même tellement séduisante qu'on peut être tenté de la considérer comme une solution nouvelle, susceptible d'être retenue pour l'équipement des stations futures, en compétition avec les prototypes décrits précédemment (voir ci-dessus). Il faut cependant remarquer qu'elle implique que le limnigraphe modifié ait une sortie en rotation (cas général des limnigraphes à flotteur, et éventuellement de certains équipements à insufflation de gaz, moyennant de légères modifications), ou émette des impulsions.

Lorsque le limnigraphe à modifier fournit une rotation en sortie, on peut aussi envisager de coder cette information grâce à un compteur-décompteur imprimant ; nous nous efforçons de faire progresser cette technique, présentant les mêmes caractéristiques générales que celles du codeur magnétique adjacent, mais permettant de plus un contrôle visuel aisé. Cette solution du compteur imprimant, retenue pour les prototypes Neyrpic et SFIM, pourrait donc également s'adapter aux équipements classiques et devenir aussi une solution complète.

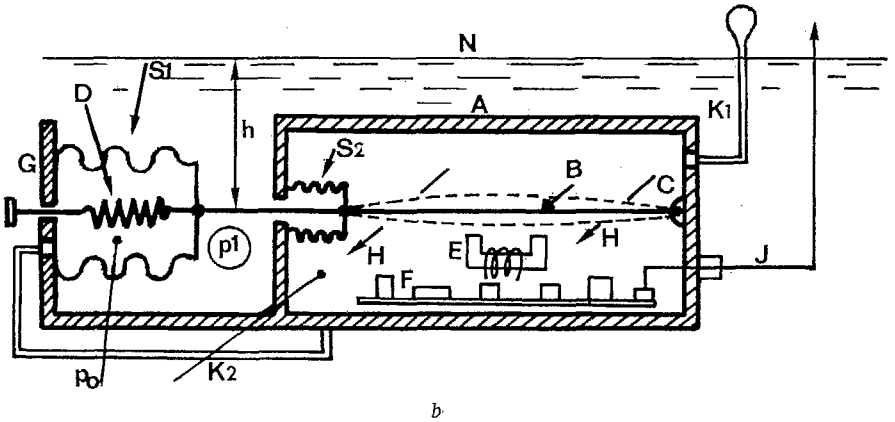
L'équipement limnigraphique est entièrement nouveau. Nous sommes là dans le domaine de la recherche technologique (voir p.701). Celle-ci ne saurait être rigoureusement dans ses conclusions, car il n'est pas possible, pour d'évidentes raisons d'homogénétité et de fiabilité, de toujours adopter le dernier modèle sorti, mais il n'est pas non plus possible de conserver indéfiniment des équipements manifestement périmés.

Le choix ne pourra intervenir que lorsque les essais, qu'ils soient satisfaisants ou non, auront été assez concluants, mais nous pouvons d'ores et déjà affirmer que nous nous attachons particulièrement aux solutions qui minimisent le coût de l'infrastructure et facilitent la mobilité du réseau (pays en voie de développement ; stations tertiaires mobiles par nature).

Les recherches technologiques ont de ce fait été nettement orientées, d'une part, sur la voie classique de l'insufflation de gaz, avec des innovations dans le domaine de l'enregistrement (voir ci-dessus : "Le limnigraphe à insufflation de gaz des établissements Neyrpic") ou dans celui de la mesure manométrique (voir ci-dessus : "Les prototypes mis au point par la Compagnie des compteurs"), et d'autre part sur la voie nouvelle de la prise de pression par jauges de contrainte (voir ci-dessus : "Le limnigraphe à insufflation de gaz" et "Le limnigraphe à prise de pression par jauge de contrainte de la SFIM").



a



b

Figure 1 – *Prototypes CDC (selon le plan n° S155 N2 du 23 mars 1970.*
a. Manomètre à ruban vibrant pour mesure de niveau par la méthode d'insufflation.
b. Manomètre à ruban vibrant, version immergeable pour mesure directe d'une hauteur d'eau.
A. Enceinte étanche ; B. Ruban vibrant ; C. Point fixe, ajustable ; D. Ressort de faux zéro ; E. Bobine d'excitation ; F. Amplificateur ; G. Point fixe ; H. Champ magnétique permanent (perpendiculaire au plan de la figure) ; J. Signal de sortie ; K. Arrivée d'air d'insufflation à débit constant ; K1 et K2. Tubes d'équilibre atmosphérique ; N. Niveau à mesurer ; p_0. Pression atmosphérique ; $p1$. Pression à mesurer, correspondant à la hauteur h ; S1. Soufflet de mesure ; S2. Soufflet d'étanchéité ; L. Buse d'insufflation.

Précisons enfin qu'on dispose actuellement d'un appareil moderne, le limnigraphe OTT 20.061 (bande perforée en code Télex et témoin visuel), d'autant plus intéressant que ses mécanismes d'enregistrement et de perforation sont les mêmes, que le capteur soit un ensemble classique "à flotteur et contrepoids", comme dans le cas actuel, ou constitué par d'autres équipements, notamment à insufflation de gaz ; un ensemble polymesures est d'ailleurs étudié par la société OTT. C'est là un exemple parfait de l'absence de solution de continuité entre les stades d'étude et de commercialisation ; rappelons que le limnigraphe OTT 20.061 était il y a peu de temps un prototype résultant de l'adjonction, à la suite de nos remarques, d'un témoin visuel au modèle 20.060, déjà commercialisé auparavant [2].

CONCLUSION

Il ne saurait s'agir ici de conclure définitivement sur les vertus de telle ou telle combinaison d'équipements, et cela ne sera d'ailleurs jamais possible . . . heureusement !

Par contre, en faisant ce tour d'horizon malgré tout assez large, nous espérons avoir rendu sensible la coordination qui caractérise les recherches technologiques exposées ci-dessus avec quelque détail. Cet aspect nous paraît particulièrement digne d'attention, car il constitue un exemple de collaboration permanente et totale entre des organismes publics et des industriels dynamiques, en même temps que d'entente entre des services administrativement très différents, mais que la technique rapproche. Cette collaboration et cette entente ont déjà donné quelques fruits, mais heureusement la récolte n'est pas terminée.

Enfin, il nous paraît utile d'insister sur le fait que les équipements limnigraphiques, s'ils font partie d'une chaîne de gestion, font partie d'un ensemble plus vaste encore, préoccupé de l'appréciation permanente de la quantité, mais aussi de la qualité des ressources en eau. Cette extension s'impose d'ores et déjà sur le seul plan de l'hydrologie (association limnigraphes-pluviographes) et elle doit être à l'origine d'une coordination plus vaste avec d'autres disciplines.

BIBLIOGRAPHIE / BIBLIOGRAPHY

1. Dubreuil, P. Les conceptions nouvelles dans le domaine du limnigraphe en 1968 : dispositifs actuellement utilisables, prototypes et projets. *Cahiers de l'ORSTOM,* série "Hydrologie", vol. V, n° 4, 1968.
2. Hlavek, R. Conceptions nouvelles en matière de mesures limnimétriques sur les cours d'eau. *La houille blanche,* n° 5, 1969. (Communication présentée au Comité technique de la Société hydrotechnique de France, le 21 mars 1969.)
3. André, H., Fleuré, J. Appareillage pour l'enregistrement, la télémesure et le traitement de l'information dans le domaine des stations de jaugeage et des stations usines. *La houille blanche,* n° 5, 1969. (Communication présentée au Comité technique de la Société hydrotechnique de France le 21 mars 1969.)

Measuring devices in stationary and mobile control stations for the supervision of rivers, shown by the example of the Lippe and Emscher rivers

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Abstract. Measuring devices have to be used for the continuous or temporary control of river water as well as for very variable waste-water inlets. The extent and type of the measurements depends on the importance of the watercourse and economy, on the quality demands on each watercourse and on the waste-water load. This paper covers the experience of nearly 20 years with a network of co-ordinated control stations and describes ways of modern instrumentation for stationary and mobile control stations which have proved efficient in the past.

DISPOSITIFS DE MESURE UTILISES PAR LES STATIONS DE CONTROLE FIXES ET MOBILES POUR LA SURVEILLANCE DES RIVIERES : EXEMPLES DE LA LIPPE ET DE L'EMSCHER

Résumé. Les appareils de mesure ont été installés en vue de contrôler en permanence ou temporairement les eaux en général, ainsi que le débit des eaux usées les plus importantes. L'importance et le type des mesures sont fonction de l'intérêt que présentent les eaux pour l'homme et l'économie, des exigences de qualité imposées à l'eau ainsi que de la charge en eaux usées.

L'auteur expose les résultats de travaux effectués depuis près de vingt ans sur un réseau de postes de contrôle coordonnés. On présente à ce sujet les moyens modernes éprouvés dont sont équipées les stations de contrôle fixes et mobiles.

APARATOS DE MEDIDA PARA CONTROL DE ESTACIONES MÓVILES Y FIJAS PARA SUPERVISIÓN DE REDES. EJEMPLO DE LOS RÍOS EMSCHER Y LIPPE

Resumen. Los aparatos de medida se han instalado con objeto de controlar en forma permanente o temporal los caudales, y también los caudales de aguas residuales. La importancia y el tipo de medidas son función del interés que tienen para el hombre y la economía, y también por la calidad necesaria y la influencia de aguas residuales.

En la presente comunicación se presenta la experiencia de unos veinte años con una red de estaciones de control coordinadas, y se describen los equipos con instrumentos modernos para estaciones de control fijas y móviles que han dado resultados satisfactorios en el periodo citado.

ИЗМЕРИТЕЛЬНЫЕ УСТРОЙСТВА В СТАЦИОНАРНЫХ И ПЕРЕДВИЖНЫХ КОНТРОЛЬНЫХ СТАНЦИЯХ ДЛЯ НАБЛЮДЕНИЯ НА РЕКАХ, ПОКАЗАННЫХ НА ПРИМЕРЕ, рр. ЛИППЕ И ЭМШЕР

Резюме. Измерительные устройства должны обеспечивать непрерывный или временный контроль речной воды, а также водовыпусков, сбрасывающих загрязнения. Объем и тип измерений зависят от важности водотока для потребителей и экономики, от требований, предъявляемых к качеству соответствующего водотока и от загрязняющих веществ. В докладе освещен опыт почти 20 лет эксплуатации сети координированных контрольных станций и пути современного оснащения стационарных и передвижных контрольных станций, оправдавших себя в прошлом.

To secure the supply of water, the most important raw material for life and the development of industry and human habitation, is often more a question of quality than of quantity. Due to the steadily increasing load in our watercourses of more or less thoroughly clarified waste waters, the maintenance of the necessary water quality can be reached only by good control of watercourses and their more important inlets. Two large Water Engineering Associations in the Ruhr District were established at the beginning of this century to supervise the different water engineering duties in the river areas of the centres of industrial development. It soon became evident that the usual methods of water control by sampling and laboratory examination did not suffice technically or economically to monitor the watercourses and their more important inlets in conformity with the steadily increasing loads. For a number of years, Lippeverband and Emschergenossenschaft have operated under joint management, developing and designing special control stations, the details of which will be described in this paper, taking as an example the water control of the Lippe river.

The core of the supervision system of Lippeverband, the details of which are shown in Figure 1, is the control stations put up 18 years ago at three particularly important points, i.e. the mouth of the Seseke brook into the Lippe river, in the area of Marl and at the mouth of the Lippe river. At these three points, the essential properties of the Lippe water are continuously recorded. Furthermore, the Marl control station (Fig. 2) supervises the main inlet of the effluents from the Hüls Chemical Works and the mouth of the Sickingmühlen brook. The Seseke Mouth control station supervises the water of the Seseke brook. In addition, four more stations were established to control large inlets of industrial effluents, viz., in Untrop, Marl, Dorsten and Bocholtwelmen. This means that these stations control, besides the three major points on the Lippe, seven important inlets of waste waters and effluents. At all these points, water samples are taken continuously using the apparatus shown in Figure 3. These samples are kept in storage vessels (Fig. 4) for at least one week in order to have samples of the respective waste waters available for examination or for evidence in case of possible trouble in the Lippe or in the Rhine.

As it is not possible to examine in the laboratory all the samples taken for noxious matter, an indicator recording reliably the effect of the water on biological life was installed. For this purpose, the control stations were equipped with fish tanks through which river water or, in case of inlets, waste water and/or a known mixture of waste water with dechlorinated mains water or river water constantly flows (Fig. 5). The use of different dilutions during the control of inlets enables the determination of the degree of toxicity in case of possible toxicity peaks. Generally, the selected dilutions are 1/10 to 1/20, at the most, of the effective mixing ratio in the drainage water, with the result that there is still sufficient security before damage occurs in the river proper. The main advantage of this very simple fish test is that it covers the superimposition of toxic effects which cannot always be determined from the specific, continuously measured or laboratory concentrations of the various load components. If the fish show adverse effects, the stored samples are available to discover the reasons for damage by relevant chemical and biological examinations. These fish tests assume great importance for industrial inlets with frequently changing and new load components. According to the experience of the Lippeverband, 4 to 8 cm long golden ides (= orfes) proved to be very satisfactory in the control stations as well as in the laboratory tests.

If the fish test is to determine only the toxic effects of the water, as is generally the case for inlets, all other factors such as oxygen, temperature, light and food have

to be kept at the optimum levels. For river water examinations only, however, it is advisable to operate adjacent trial experiments with or without aeration of the aquaria. Attention must be paid to the fact that some fish become accustomed to some components of the water and react less sensitively to sudden increases in concentrations than fish not adapted to such water. It is thus advisable to exchange such fish after one or two weeks. The testing of samples by the fish tests—a routine practice in all control stations of Lippeverband and Emschergenossenschaft—has proved to be good practice. As the fish generally react much more sensitively to toxic loads than warm-blooded animals, which take in water only through the intestines, the fish test gives safety against most of the sudden toxic loads to warm-blooded animals and thus also man.

In addition, the two Associations developed and tested in the last decades a number of automatic measuring devices to determine specific properties and load factors of the water. In comprehensive tests, many devices to record the oxygen content, turbidity and salt loads were studied. Further devices, a few of which were also installed in control stations, are based on the developments of the engineers of the two Associations, such as the device for the continuous measurement of sediments (Fig. 6) and the device with stained filter paper (Fig. 7). The additional equipment of the control stations with special devices depends, of course, on the individual requirements of the control point and the types of loads occurring there. An example of such equipment is the control station at the mouth of the Lippe, i.e. at the point where it is necessary to record the more important properties of the Lippe water prior to flowing into the Rhine. The flow diagram of this station is shown in Figure 8 and some of its equipment in Figures 9 and 10. The following parameters are continuously measured at this point: oxygen content, water temperature, air temperature, conductivity, pH, turbidity and structure of solid matter (stained filter paper).

In addition, continuous fish tests are carried out for the control of toxicity and two-hour mixture samples are taken and stored for seven days. These mixture samples serve for the analysis for special load factors (e.g. oil, phenols, heavy metals) which is sometimes necessary and for the daily determination of total activity. The river water is sucked by the pump of about 10 m^3 capacity and pushed into the overflow tank. No degassification must be allowed during pumping of the water to the devices and the flocculent structure of the settlable matter must not be disturbed by the moving water. From the overflow tank, a certain volume is conducted in the supply line with the required velocities to the various measuring devices and/or aquaria. Beyond that, the water flow is directed every two minutes via a switch control into the interceptor, from which the mean samples are taken every two hours. The cost of the equipment with these instruments amounts to about 33,000 DM plus the cost for building and the supply of water. Of course, the watercourse must be checked prior to the building of the control station to see whether the water is evenly mixed over the cross-section of the pipes, so that the samples taken give a representative picture of the actual load conditions.

In a similar form, the inlets to and, above all, the outlets from the larger purification plants of the Associations are continuously supervised. The measuring devices used at these points have the duty of controlling the operation of the plant and fulfil certain functions, i.e. the setting of the oxygen contents in the aeration tanks. Figure 11 shows the measuring and control centre of the purification plant of Emschergenossenschaft where the waste water equivalent to a population of about 200,000 is biologically treated. Also at this plant, the outlet of which flows directly

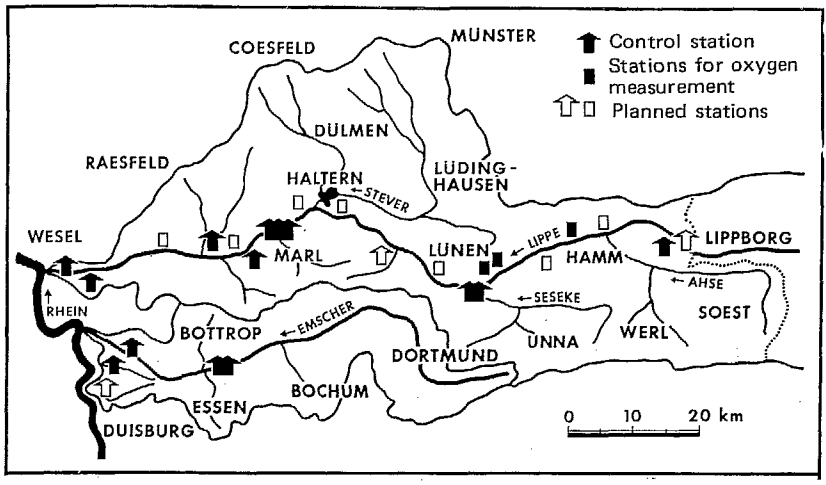


Figure 1 – *Control stations in the regions of the Lippeverband and Emschergenossenschaft*



Figure 2 – *Marl-Hüls control station*

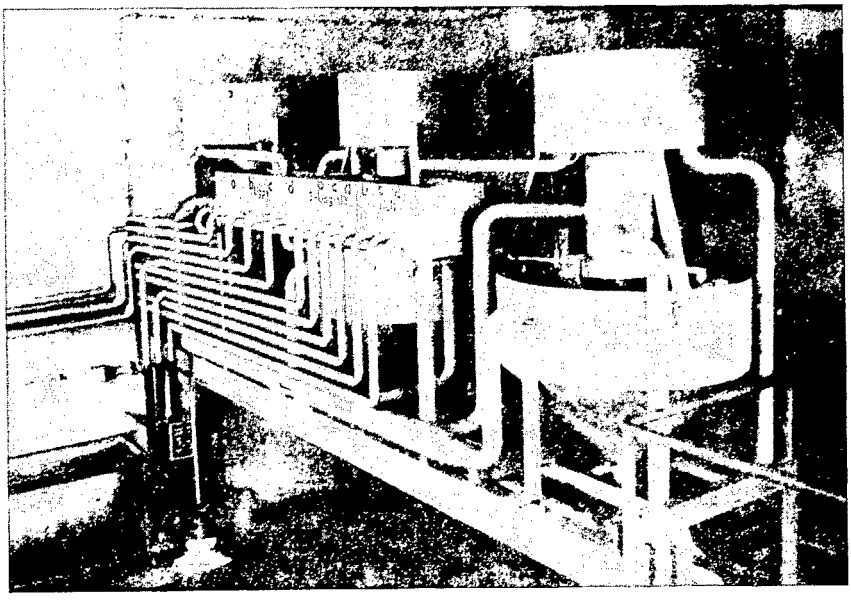


Figure 3 – *Equipment for taking samples in the Marl-Hüls control station*

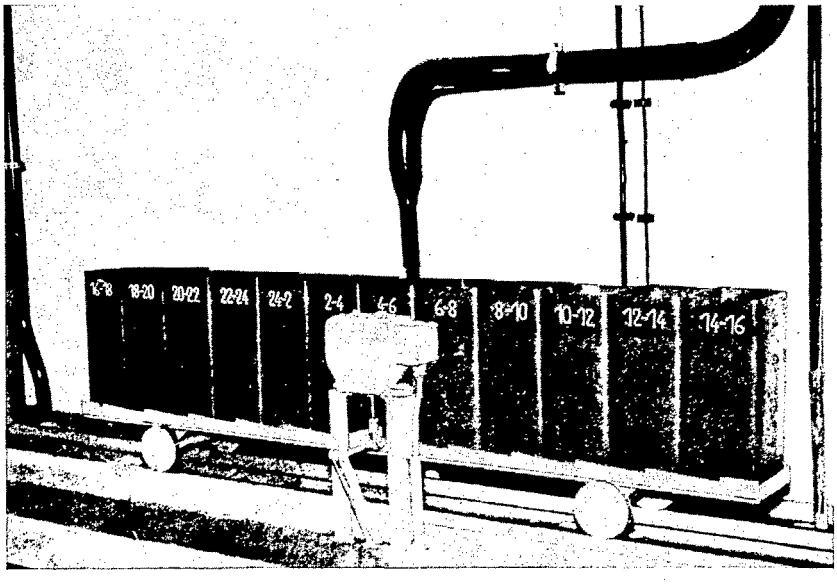


Figure 4 – *Sample storing vessels in the Marl-Hüls control station*

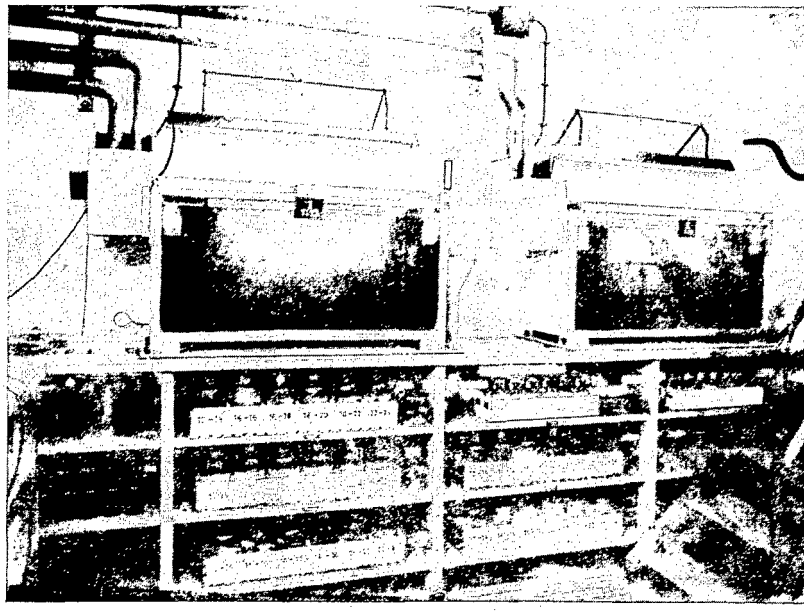


Figure 5 – *Fish tanks in the Marl-Hüls control station*

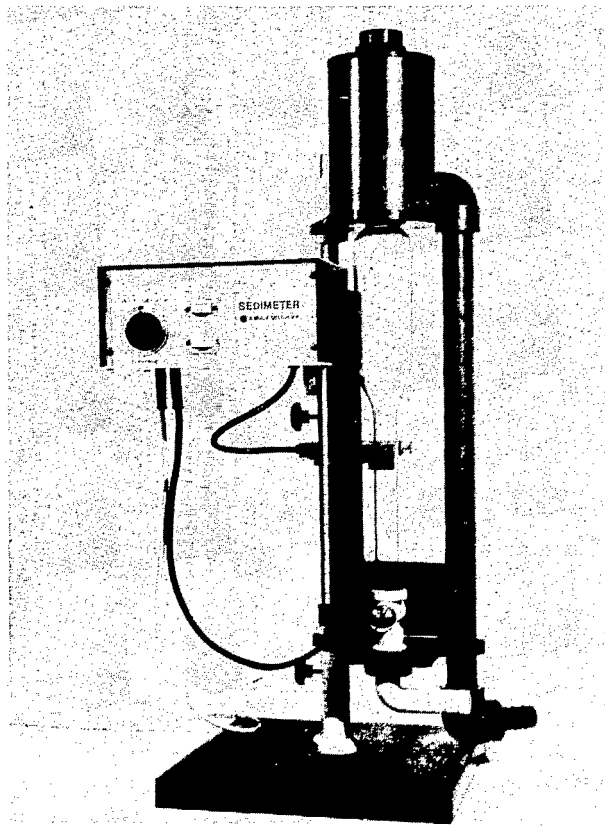


Figure 6 – *Sedimeter for automatic measurement of sediments*

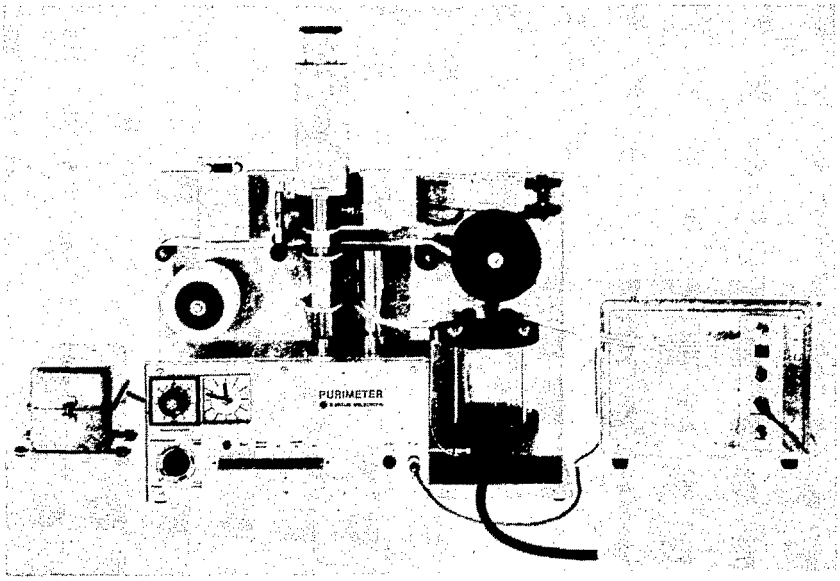


Figure 7 – *Stained filter paper device*

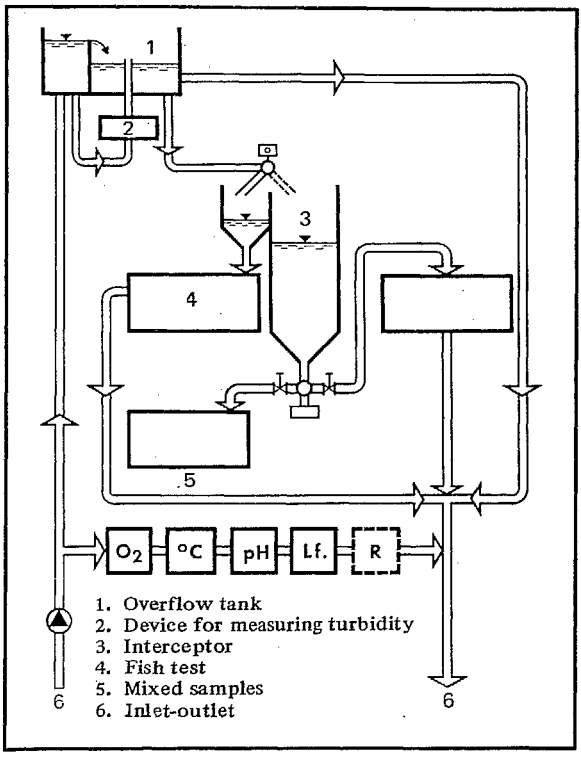


Figure 8 – *Flow diagram of the Wesel control station*

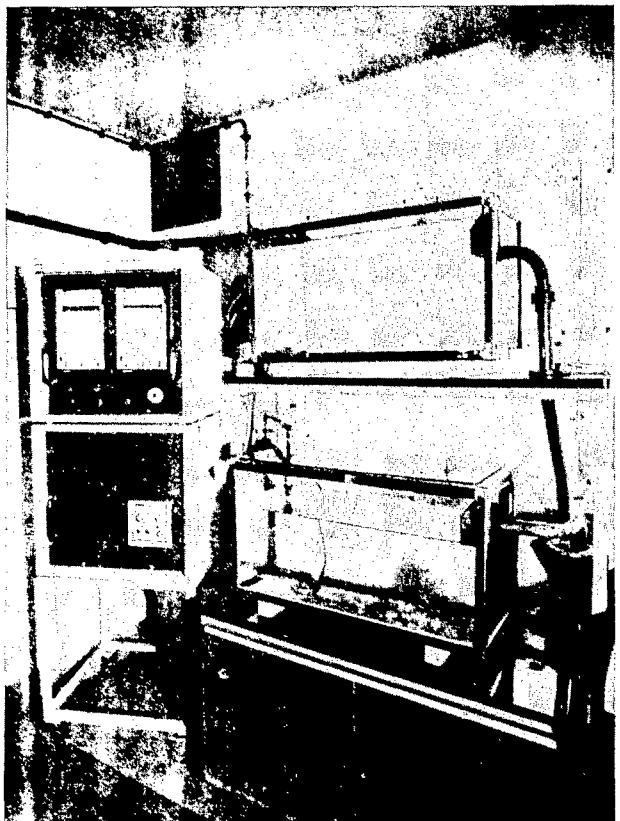


Figure 9 – *Fish tank and recorder of measured data at the Wesel control station.*

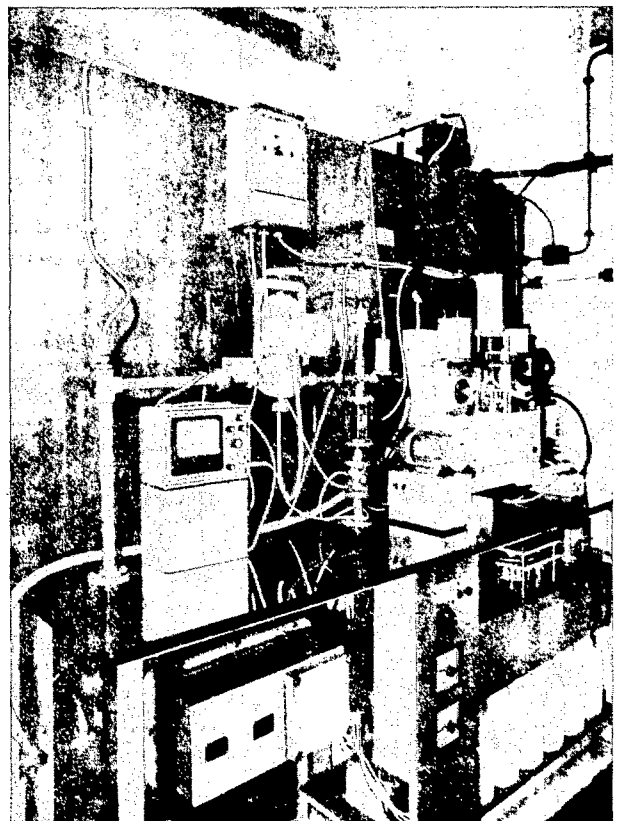


Figure 10 – *Measuring devices and equipment for taking samples at the Wesel control station*

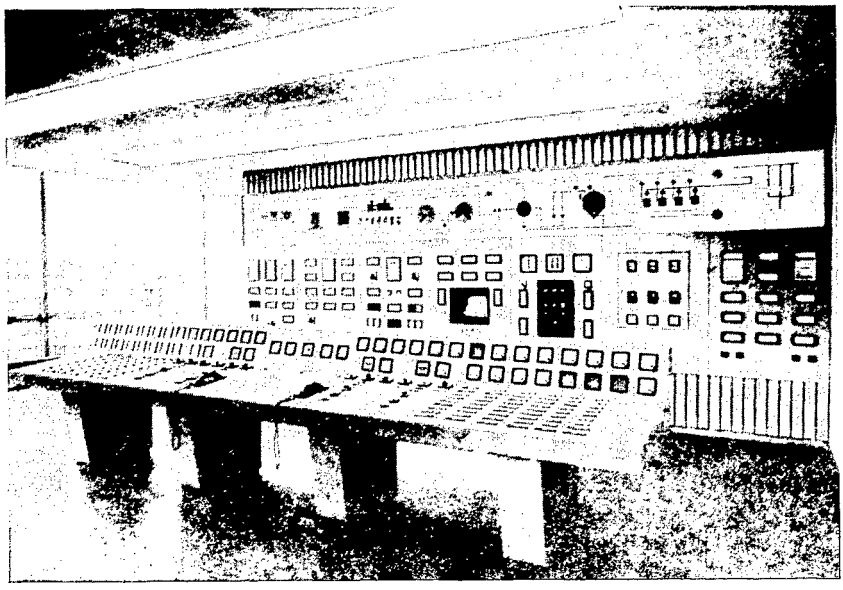


Figure 11 – *Control centre at Duisburg-Kleine Emscher purification plant*

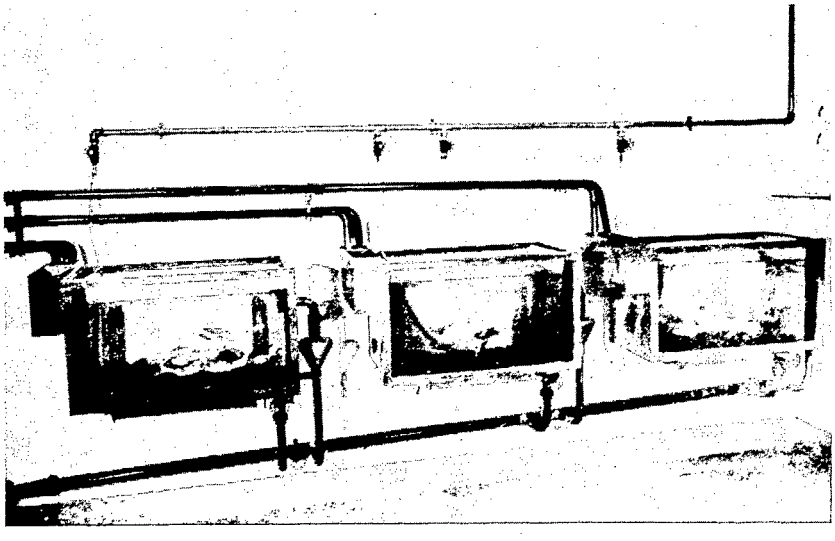


Figure 12 – *Fish tanks in the control station of the Duisburg-Kleine Emscher purification plant*

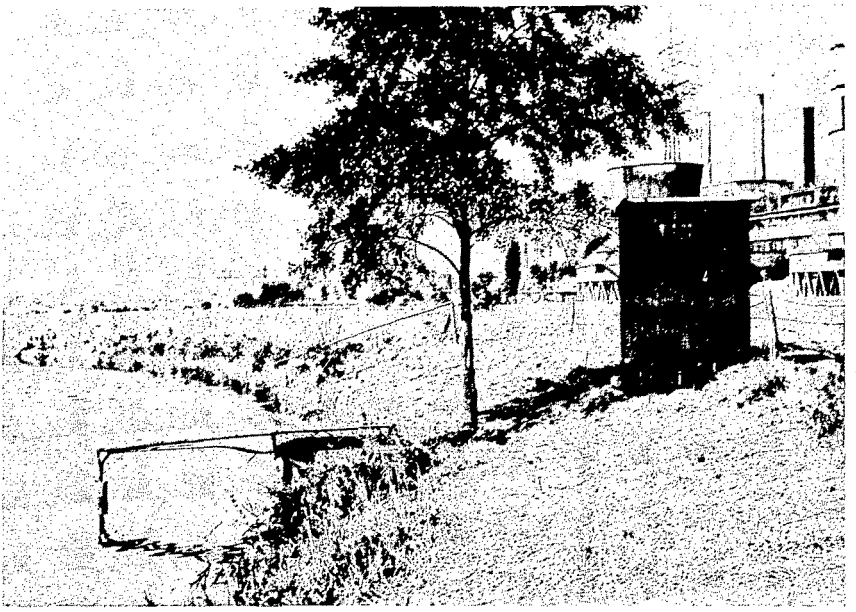


Figure 13 – *Measurement of oxygen contents*

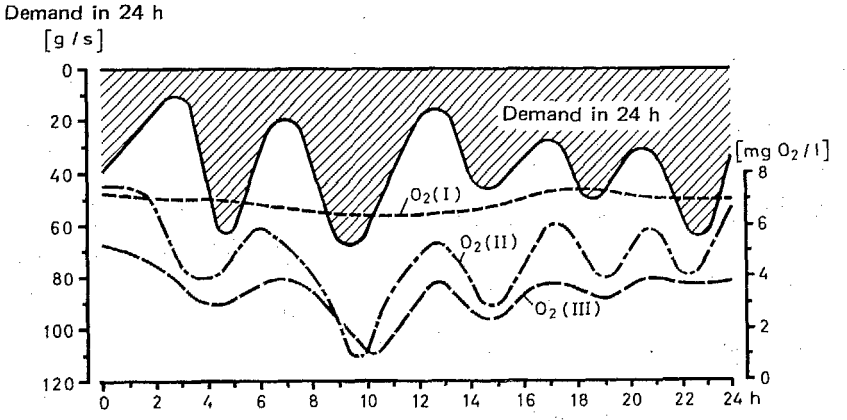


Figure 14 – *Oxygen demand.*

into the Rhine, a sample of the purified water is directed continuously, for control purposes, through fish tanks (Fig. 12).

In addition to the control stations with the above-mentioned full equipment, Lippeverband also installed other measuring stations along the Lippe of a lighter and mobile type, where temperature and oxygen contents of the water as well as the temperature of water and air are registered at important points on the river (Fig. 13). The measured values are retained by multi-colour records. The cost of equipment for such stations comes to about 7,000 DM. It is planned to establish still more stations of this mobile type, according to need, along the Lippe.

Figure 14 shows the importance of not restricting the oxygen investigation at certain points of the river to random samples, but to record the curve continuously. The corresponding oxygen values of three points of a certain section of the Lippe river are plotted in this figure during low flow periods. The inlet of a large industrial plant, the effluents of which are, at present, still insufficiently clarified, is situated between the measuring station I and Station II. The effects of this inlet can be seen from the curve of oxygen consumption during 24 hr, which is also plotted in the figure adjusted to the flow times. Whereas the oxygen curve of measuring point I is nearly horizontal during the 24 hr, the curves from the measuring points II and III show conspicuously the effects of the inlets with the intermittent loads and give an impression of the erroneous conclusions which can be drawn from the taking of random samples alone. The curve shows the result of a measurement during the winter. During the summer months, conditions are still more distorted by the effect of the biogene aeration during the day and the corresponding oxygen consumption during the night.

The third element in the control system of Lippeverband and Emschergenossenschaft are mobile measuring stations which, if required, can be used within short notice (about 2 hr) at the necessary points (Fig. 15). The essential parts of this

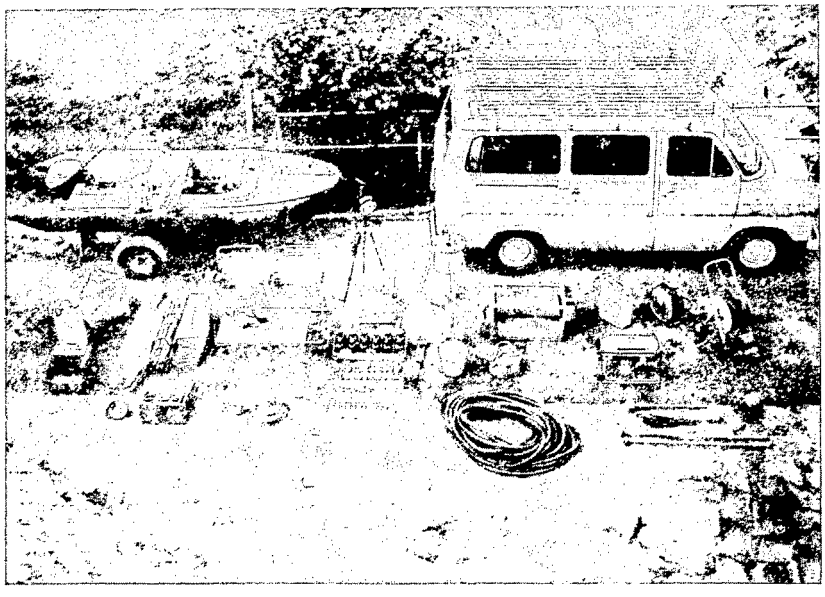


Figure 15 – *Equipment of mobile measuring stations*

equipment are, apart from the emergency power equipment and the intake pumps, through-flow aquaria, certain devices for the continuous measurement of important parameters and a set of mobile apparatus (also developed by engineers of Lippeverband for the taking of samples), a boat equipped with an outboard motor and a tent.

Needless to say, the use of a number of stationary control stations and some further mobile control stations for continuous measurements do not make individual measurements unnecessary. Continuous measurements should and must be carried out only where the importance of the river and/or the inlet require such a measure. On the other hand, such stationary measuring points, provided they are designed in good time in connexion with larger inlets and/or clarification plants, can be put up at rather low cost and require, for permanent operation, only a fraction of the expenses needed for an equivalent control of another type. In view of the trend for automatic measuring devices to improve further and become cheaper in the course of development, while the expenditure for specialists as required for individual tests are currently increasing, the future supervision and control of our watercourses should move still more towards the automatic devices and stations. It is hoped that the examples given in this paper will have shown that technically and economically acceptable equipment and devices are used which guarantee the sufficient control of watercourses and important inlets, with the result that serious accidents in our rivers and watercourses can be avoided or, at least, discovered in good time.

HYDRA II
automatic digital telemetering system

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Abstract. Telemetering system HYDRA II is designed to transmit, collect, record and evaluate hydrological information according to a given programme. Furthermore, depending on the results of the evaluation, it may issue signals or orders. Its working radius is about 50 km. At present it is capable of collecting precipitation and water level information. By connexion of the units under development to the system, it will be able to collect additional information of a similar nature. The information is recorded in decimal units on a paper tape. A signal is produced when a present critical instantaneous value is attained or exceeded or a change in value occurs. It can be operated either automatically or manually as and when required.

Thirty-two stations can be connected to a centre, and at the same time three digital and six analogue sensors can be connected to a station.

SYSTEME TELEMETRIQUE NUMERIQUE AUTOMATIQUE HYDRA II

Résumé. Le but du système télémétrique HYDRA II est d'assurer le transit, la collecte, l'enregistrement et l'évaluation d'informations conformément à un programme donné de mesures de paramètres hydrologiques et hydro-économiques à partir de stations situées dans le champ d'action d'un centre de mesure. De plus, en fonction de l'évaluation obtenue, ce système peut fournir signaux et ordres.

Son action s'étend dans une région d'environ 50 kilomètres. Dès à présent il peut rassembler les informations concernant les hauteurs de précipitation et les hauteurs d'eau. Par simple connexion à d'autres capteurs, dont la mise au point se poursuit, il pourra collecter des informations supplémentaires. Les informations sont enregistrées en unités décimales sur une bande enregistreuse. Un signal se produit lorsque est atteinte ou dépassée une valeur instantanée critique préétablie ou lors d'un changement de valeur. Cette opération peut être déclenchée automatiquement ou manuellement à volonté.

On peut connecter 32 stations au centre de mesure et chaque station peut comprendre 3 capteurs numériques et 6 analogiques.

SISTEMA DE TELEMEDICIÓN DIGITAL AUTOMÁTICO HYDRO II

Resumen. Los fines del Sistema de Telemedición HIDRO II son transmitir, recoger, registrar y evaluar la información de acuerdo con un programa dado sobre acontecimientos hidrológicos y de economía del agua dentro del área del centro de medición correspondiente, y además, emitir señales y órdenes, en función del resultado de la evaluación.

Su radio de acción es de unos 50 km. Actualmente es capaz de recoger información sobre altura de precipitación y nivel de agua. Por conexión simple de las unidades del sistema, éste es capaz de recoger información adicional de otra naturaleza. La información se registra en unidades decimales en una cinta de papel y se produce una señal cuando se alcanza o excede un valor instantáneo crítico preestablecido, o aparece un cambio de medida. Se puede poner en funcionamiento en forma automática o manual.

Se pueden conectar a un centro 32 estaciones, y al mismo tiempo se pueden conectar a una estación 3 detectores digitales y 6 analógicos.

ГИДРА-II — АВТОМАТИЧЕСКАЯ ЦИФРОВАЯ ТЕЛЕМЕТРИЧЕСКАЯ СИСТЕМА

Резюме. Назначением телеметрической системы Гидра-II является: передавать, собирать, записывать и оценивать информацию соответственно с заданной программой на гидрологические и водноэкономические события на гидрометрических станциях, которые, находясь в районе измерительного центра, кроме того, зависят от результатов оценки, и выдавать сигналы (команды).

Радиус действия телеметрической системы составляет около 50 км. В настоящее время представляется возможным сбор данных об атмосферных осадках и сведений об уровне воды. Благодаря простой связи элементов, при развитии их в систему, становится возможным собирать дополнительную инфорзмацию иного характера. Информация записана в десятичных единицах на бумажную ленту. Сигнал срабатывает, когда достигнуто или превышено заранее установленное критическое мгновенное значение или изменение в величине. Это может быть произведено или автоматически или вручную в отдельные моменты.

32 станции могут быть объединены в центр, и в то же время 3 цифровых и 6 аналоговых датчиков могут быть связаны со станцией.

OBJECTIVE

The telemetering system HYDRA II is designed to transmit, collect, record and evaluate hydrological information according to a given programme from up to 32 measuring stations within a 50 km radius of the centre; furthermore, as a result of the evaluation, it may issue signals or orders. By information is understood a numerical value for such variables as: water level, water discharge, precipitation, temperature, air humidity, wind speed, soil moisture, snow density equivalent and sluice-board position.

By connexion of further sensors under development, the system could collect additional information. The information is recorded in decimal units on a paper tape on which is also recorded the exact date to the minute and the numbers of the measuring station and the sensor. A warning signal is produced when a preset critical instantaneous value is attained or exceeded or a change in value occurs. It can be operated either automatically, according to a time programme, or manually as and when required.

CONSTRUCTION

The system consists of a measuring centre and measuring stations. Because of the distances and the ground conditions occurring in practice, radiotelemetry is used between the centre and the stations (Fig. 1).

The construction of the radio telecommunication system is as follows. Since the measuring system is of digital design, all information transmitted within the system appears in the form of d.c. pulse series, the number of which is proportional to the value of the measured quantity. Transmission is effected between centre and stations by a radio link at a high carrier frequency to reduce the required transmitter output power and with frequency modulation in order to avoid interference. For radio transmission the d.c. pulses are converted into audio-frequency pulses which modulate the carrier frequency. In the receiver the audio-frequency pulses are converted

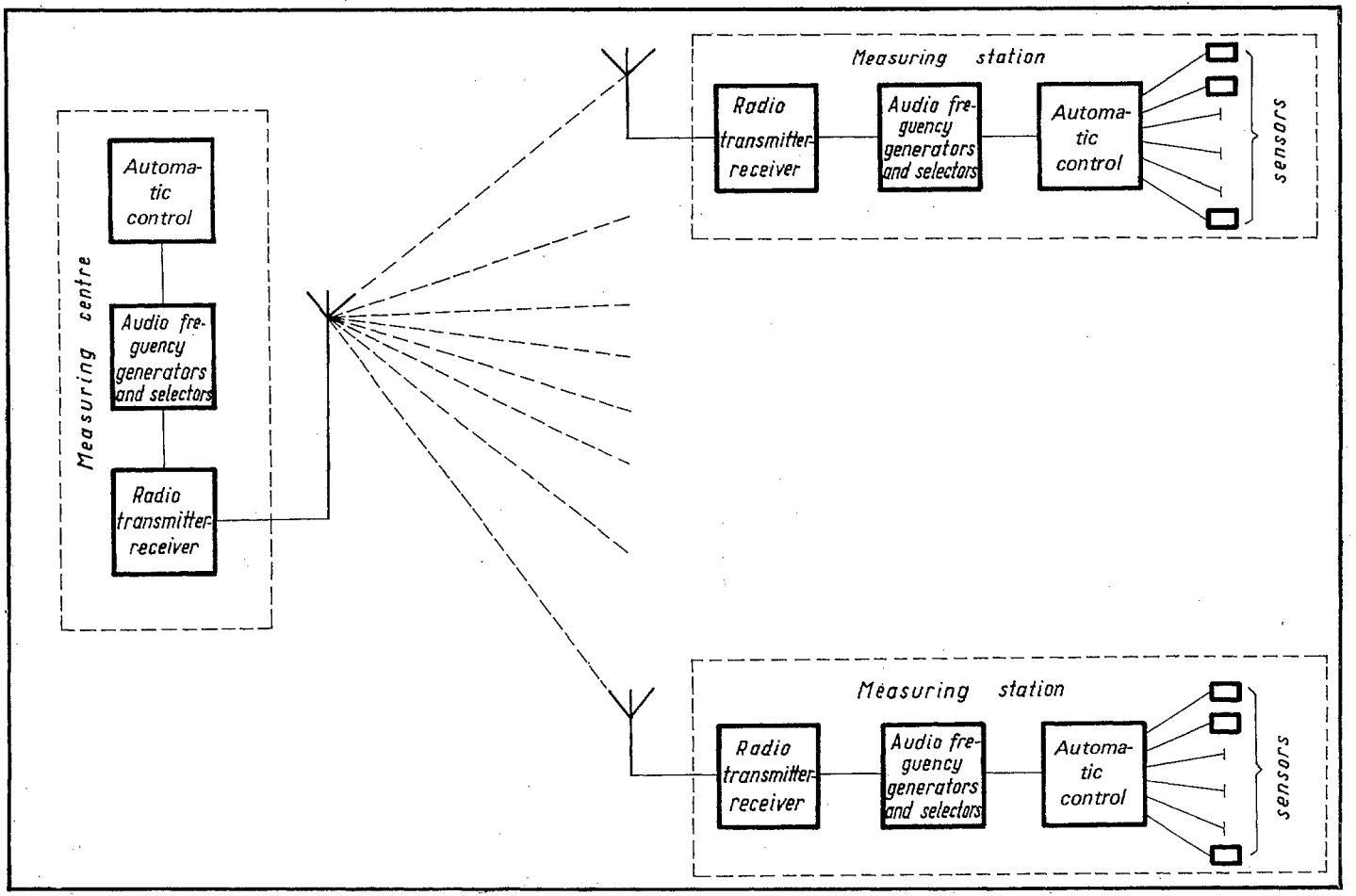


Figure 1 – *Block diagram of the telemetering system*

again into d.c. pulses. One duplex carrier frequency pair is used for the whole system, whereas the modulating audio-frequencies differ from station to station. The data transmission channels are essentially stationwise individual audio-frequency channels.

The measuring system consists of the following functional units (Fig. 1):

Centre
automatic equipment (including line printer, schematic display board, operator's board);
transmission channel unit;
audio-frequency generators and selectors;
radio transmitter-receiver.

Measuring station
transmission channel unit;
radio transmitter-receiver;
audio-frequency generators;
automatic control equipment;
sensors.

The central control equipment always initiates operation (measuring, recording, evaluation) according to the cyclic time programme, i.e. the system is of a central interrogation design.

The sensors at the different stations are transducing the information. The individual sensors have either a digital or an analogue output; in the latter case it is converted to a digital output by a single analogue-to-digital converter in the station's automatic control equipment. In the case of sensors for measurement of precipitation and water level, the output is in digital form. Depending upon the requirements, measuring networks of various capacities can be built from the above units. The upper limit of the capability is determined by the fact that a maximum of 32 stations can be connected to a centre; at the same time not more than three digital and six analogue sensors can be connected to a station. Within these figures the upper limit of the system depends on the number of stations, the number and type (digital or analogue) of the individual sensors used at the different stations, the evaluation to be done by the centre, the number of sensors to be watched from the point of view of critical instantaneous values and changes of value, the time programme, the cycle time and speed of operation.

APPLICATION

The following are the different fields of application of HYDRA II:

water governing and water economics systems for internal waterways;
water economics authority verifying systems;
irrigating systems;
water supply systems;
data collecting systems serving hydrological research.

With the aid of the telemetering system measurements can be made independently, to a great extent, of the measuring personnel. Measurements can be effected in respect of their location, time and frequency according to a programme corresponding to the purpose and to the nature of the factor to be measured. Measurements can be co-ordinated in time within a given area; practically, the distance between measuring station and centre can be bridged without a time delay, considerably

increasing the data transmission speed. Measuring results can be evaluated either simultaneously or at a later date; manually or automatically. In the latter case, instead of a line printer punched paper-tape should be used.

FURTHER DEVELOPMENT

HYDRA II is still being developed, and the improvements are as follows:

simplification of installation and operating conditions;
enlarging the variety of sensors;
reducing the electric power consumption; solving the data transmission problem for communication by wire with line terminal units.

Further variants of the HYDRA system are being developed, such as: widening the services of the centre, including complex evaluation of the information and issue of remote control instruction, and designing the variants corresponding to the most important practical applications.

The Research Institute for Water Resources (VITUKI) developed the system and produced the first experimental prototype, and the Institute for Electric Automation improved the design and manufactured the system.

Water-quality monitoring and data transmission

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Abstract. The evolution of automatic water-quality monitoring within the Water Resources Division of the United States Geological Survey is briefly reviewed, and the automated water-quality monitoring system now in use by the Geological Survey is described. The description includes all principal components from the sampling technique to the computer-processed data compilation. A description of the digital telemetry system most frequently used by the Geological Survey is also included.

The experience gained from operating a nation-wide network for a number of years is presented. These experiences have led to the adoption of certain techniques and operating procedures. Those procedures and techniques pertaining to training of field personnel, construction of on-site facilities, calibration of sensors, establishing maintenance schedules, and processing of data are discussed.

CONTROLE DE LA QUALITE DE L'EAU ET TRANSMISSION DES DONNEES

Résumé. L'évolution du contrôle automatique de la qualité de l'eau au sein de la Division des ressources en eau du Service géologique des Etats-Unis est brièvement passée en revue et l'on donne une description du système de contrôle automatique de la qualité de l'eau : principaux éléments du système depuis la technique de la prise d'échantillons jusqu'à la compilation des données traitées sur ordinateur. On décrit également le système télémétrique digital le plus fréquemment utilisé par le Service géologique. L'auteur fait part de l'expérience acquise en opérant pendant de nombreuses années sur un réseau à l'échelle nationale. Cette expérience a conduit à adopter certaines techniques et certains modes opératoires.

Sont indiquées également les méthodes et les techniques suceptibles d'être utilisées pour la formation du personnel sur le terrain, pour l'installation d'équipements pratiques sur les lieux, pour l'étalonnage des capteurs, pour l'établissement de programmes d'entretien et le traitement des données.

CONTROL DE CALIDAD DEL AGUA Y TRANSMISIÓN DE DATOS

Resumen. En la presente comunicación se pasa una breve revista a la evolución de los dispositivos automáticos de control de calidad de agua existentes en la División de Recursos de Agua del Unitel States Geological Survey y se describe el sistema de control automático actualmente utilizado.

La descripción incluye todos los componentes principales desde la técnica de toma de muestras a la recopilación de datos y proceso por ordenador. También se incluyen una descripción del sistema de telemetría digital más frecuentemente utilizado por el Geological Survey. Se muestra la experiencia obtenida con el funcionamiento de una red a escala nacional durante cierto número de años. Estas experiencias han conducido a la adopción de unas ciertas técnicas y determinados procedimientos de funcionamiento. Dichos procedimientos y técnicas se refieren al entrenamiento de personal de campo, construcción de instalaciones localizadas, calibración de detectores, establecimiento de planes de mantenimiento, y proceso de datos.

КОНТРОЛЬ ЗА КАЧЕСТВОМ ВОДЫ И ПЕРЕДАЧА ИНФОРМАЦИИ

Резюме. Дано краткое описание расчета автоматического контроля за качеством воды, проведенного отделом водных ресурсов Геологической службы США, и описана система автоматического регулирования качества воды, которая в настоящее время используется Геологической службой. Описание включает все основные компоненты от методики взятия проб до накапливания данных, обработанных на компьютере. Также включено описание цифровой телеметрической системы, наиболее часто используемой Геологической службой.

В докладе освещен опыт, полученный при управлении национальной сетью за ряд лет. Этот опыт привел к принятию определенного оборудования и методики выполнения работ. Рассмотрены методика и аппаратура, предназначенные для обучения полевого персонала, установка оборудования на месте, тарировка датчиков, получение эксплуатационных графиков и обработка данных.

INTRODUCTION

The Water Resources Division of the United States Geological Survey first began using on-site electronic monitoring in 1955. The early units were limited to sensing and recording water temperature and specific conductance. Later, the sensing of dissolved oxygen and other parameters was added, using separate instruments and recorders. In 1960 the first fully integrated monitors were being evaluated. By 1965, it had become obvious that the mass of data collected on strip charts could not economically be reduced to a usable form by manual means. A programme was then initiated to develop an integrated monitoring system which recorded data in a form suitable for automatic data processing. This programme led to a monitoring system of modular design which records water-quality data on punched paper tape.

The Geological Survey has almost 300 water-quality monitors in use across the country. Approximately one-half of these are multiple-parameter units, of which only a few are the older, analogue type. The other half consists of simpler, battery-operated units which sense only temperature and specific conductance; these record on punched paper tape, also. Developed and funded through the Geological Survey's Federal-State or municipal co-operative programme, this monitoring network has evolved from local needs and priorities, coupled with national objectives and guidance.

MONITORING EQUIPMENT

The monitoring system as used by the Survey consists of six basic sections (Fig. 1) : (1) pump; (2) sample chamber; (3) sensors; (4) signal conditioners; (5) servo programmer; (6) recorder.

The flow-through sensor chamber houses the sensors and receives a continuous flow of sample from a submersible pump located in the stream. Water flows through to all sensors in parallel compartments, as opposed to series, so that interaction between sensors is prevented. The physical configuration of the sampling module reduces accumulation of sediment, algae, and small debris brought in by the pump, and the design of the system minimizes deterioration and contamination of the sample.

The output from each sensor is fed into individual signal conditioners where the signal is amplified and, if needed, linearized. The output range of each signal conditoner is a 0-5 volt signal which is linearly proportioned to the value of the parameter. Each signal conditioner has both "zero" and "span" adjustment capability and some, such as specific conductance, chloride, and turbidity, have two or more manually selectable measuring ranges.

The servo-programmer is designed to accept signals from as many as ten signal conditioners (and sensors) and, upon a signal from a timer, to sequence the output of each of these to the digital recorder. The timer may be set to programme at a preselected time interval of either 15, 30, or 60 min.

The on-site analogue-to-digital recorder (ADR) is basically a device for encoding and digitally recording, in 16-bit binary coded decimal (BCD) form on punched paper tape, the rotational position of an input shaft.

Sensors suitable for use with the monitoring system described are available for a variety of parameters and, generally, for a particular parameter there are several sensors from which to select.

TELEMETERING EQUIPMENT

The need for current data availability has become increasingly important in the past few years. Many users of Geological Survey data require current information for efficient and effective operation. This is especially true in the area of water quality; therefore, no discussion of monitoring is complete without some discussion of telemetry.

The telemetry most frequently used by the Geological Survey is a digital system designed specifically for the transmission of hydrologic data (Fig. 2). The digital system allows the data to be received in computer-compatible form.

As data are recorded on site, they are stored in a memory bank in BCD form so that the information can be made immediately available for transmission. Recording and storage of data are initiated at a preselected timed interval by a contact closure of the timer. The water-stage recorder, ADR No. 1, is continually balanced; therefore, its value is immediately recorded. The same contact closure within the timer that activates the water-stage recorder also activates the water-quality monitor programmer. The programmer sequences sensor number one (specific conductance) into the system, allows time for the servo-drive to balance ADR No. 2 and calls for the value indicated by the sensor to be recorded. It then sequences sensor number two into the system, balances the ADR, and records the value indicated. This same procedure is repeated until all active channels have been recorded, at which time the programmer returns to the off position and awaits a new initiating signal from the timer. It should be noted here that the system will operate with only water stage and ADR No. 1, or with only the water-quality monitor and ADR No. 2. In addition, the number of parameters available through the water-quality monitor may vary from one to ten.

At the time water stage is recorded on ADR No. 1, a set of electrical contacts within the ADR, representing the recorded value in BCD code, is closed. This value is transferred into the telemeter's water stage memory location through the input addresser. The value for specific conductance is recorded on ADR No. 2 a few seconds after the recording of water stage on ADR No. 1 as a result of the lag in the servo-programmer. Its value, as represented by the set of electrical contacts in ADR

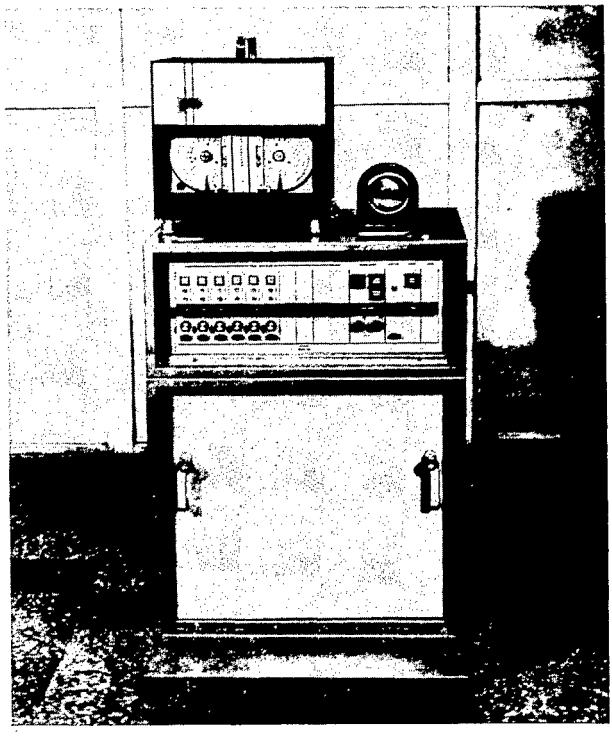


Figure 1 – *Water-quality monitor*

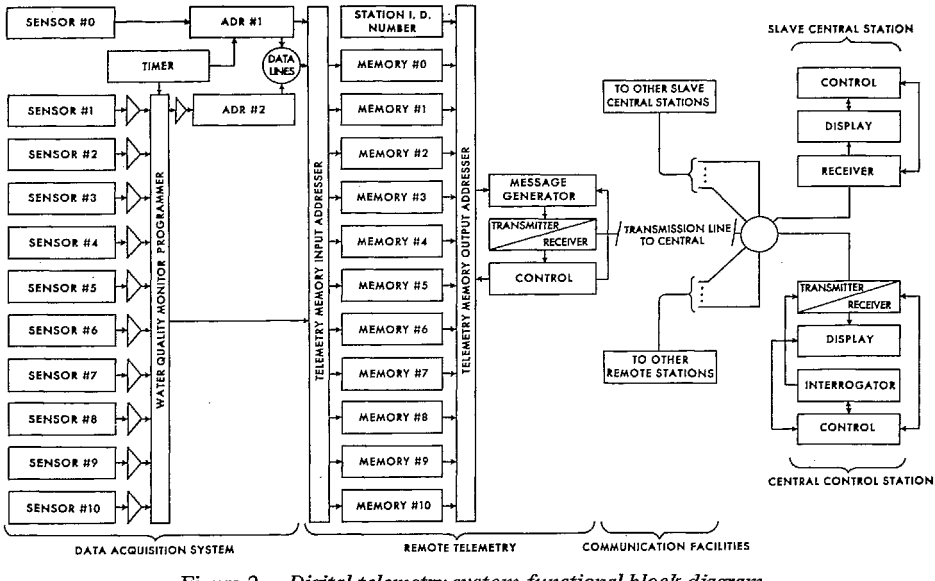


Figure 2 – *Digital telemetry system functional block diagram*

No. 2, is transferred to the specific conductance memory location through the memory input addresser. The sequence is repeated until all parameters are entered into the appropriate memory storage modules. The input addresser relies on parameter identifier output from the water-quality monitor programmer and ADR No. 1 in determining proper memory locations.

Upon completion of the recording cycle, digital data in BCD form representing the parameter measurements are stored in memory and are ready for transmitting at any time. These values will remain in the memory modules until they are erased and new values inserted in their place.

A telemetry network may consist of several remote stations reporting to a central receiving station, and each of these remote stations may have several parameters. Therefore, it is necessary to provide both station identification and parameter identification. This has been done by placing fixed identification numbers in each memory module within the memory bank. The first module always represents the station identification and is wired in such a manner as to maintain the identification number assigned to it. Each succeeding module represents a parameter and has space for 5 digits of information. In the case of water stage the first digit in its module is a 2 which is permanently fixed as an identifier, and the following four digits represent the value of water stage to the nearest one part in ten thousand. In the other individual parameter modules the first two digits are permanently fixed and represent the parameter identification number ranging from 01 to 19. The next three digits represent the value of the parameter. This arrangement allows for an accuracy of one part in a thousand.

Actual transmission of data is initiated by a timer at the master central station. The station identification code of the remote station from which data is desired is formed by the interrogator and transmitted to all remote stations. Only the station which has this particular code stored in its station identification storage module responds. It samples the value in the storage identification module, and transmits this value as a series of pulses. These pulses are received at the central station, causing a teletypewriter to type and punch out on paper tape the information transmitted.

The memory output addresser then advances to the water stage storage module and these data are transmitted and recorded. This procedure is repeated until all information from the remote station has been transmitted and recorded. The interrogator then calls for the next remote station to report and the entire process is repeated. Many remote stations can be interrogated by a single central station in this manner. In addition, slave central stations may be operated from the same system; however, they perform no control function and they serve only as data receivers.

The communications linkage may be leased line (either telegraph or voice grade), radio, microwave, or a comination of these. The rate of transmission is limited by the type of communications linage used, and the final output device. This transmission rate is controlled by plug-in modules in the telemeter transmitter and receiver. These are easily exchangeable so that a system's speed may be selected to best suit the communication linkage. Maximum rate over a telegraph grade line is 15 pulses per sec; therefore, approximately 2 sec are required to transmit one parameter. Maximum rate over a voice grade line is approximately 2,300 pulses per sec. and transmission time is reduced proportionately. Rates over both radio and microwave can be considerably faster. However, when combinations of the different types of linkages are used in a single system, transmission rates cannot be faster than the capabilities of the slowest speed linkage.

Because this is a modular system, a high degree of flexibility is obtained. For example, it is not necessary that a teletypewriter be used for recording data; any

other digital recording device can be used. Also, accessories such as digital clocks may be added. Thus, the system can be adapted to the specific needs of the user.

OPERATING EXPERIENCES

The choice of monitoring systems and accessory equipment has been made on the basis of the needs of data users, coupled with the Geological Survey's long and varied experience in the field of water-quality data collection. Similarly, a set of procedures and techniques for operating a national network has evolved.

Our experience has shown that the single most important factor in successful operation of a monitoring network is availability of qualified personnel. Thorough and adequate training is essential. Our training programme consists of individualized instruction in all phases of monitor installation and operation. When a new monitoring installation is planned, the field man who will be responsible for its operation is brought in to headquarters for instruction. There he personally uncrates and assembles his monitor for a thorough checkout. At this time he is given detailed instruction in calibration, maintenance and trouble-shooting procedures. When this phase of the training is completed, he visits the data processing unit to familiarize himself with the procedures used in the processing of the data.

The amount of usable data collected is directly related to the frequency of servicing the installation. It has been our experience that the typical monitor system needs servicing approximately once per week. Some systems may require several visits per week, while a few may need servicing only every other week.

At each visit the operator inspects the unit; performs a series of electronic tests to determine if the unit is operating properly; removes the record, if scheduled; cleans the sensor chamber and sensors; and inspects the pump and removes accumulated debris.

At very few of our multiple-parameter installations are the sensors placed directly in a stream; most locations use a submersible pump to deliver water to the sensors in a sample chamber. Although the greatest source of lost record results from pump failure, the advantages of accessibility of the sensors for frequent servicing and the additional protection from vandalism usually outweigh this disadvantage.

All sensors are calibrated by adjusting the recorder reading (signal conditioner output) to agree with the known parameter value of a solution in which the sensor is immersed. The use of portable instruments for calibrating the monitor is discouraged. The following are the recommended porcedures.

Temperature. Calibration of the temperature system is achieved by immersing the temperature probe in a solution of water and ice and adjusting the zero adjustment so that the recorder reading agrees with the temperature of the solution as indicated by a laboratory-grade mercury thermometer. The same procedure is followed with the probe in warm water. Calibration is usually made to the nearest 0.1° C.

Specific conductance. The specific conductance sensor is calibrated using standard potassium chloride solutions prepared according to instructions in *Standard Methods for Examination of Water and Waste-water,* 12th edition. At least two solutions are used, one near the full scale and another at near mid-scale. Zero is adjusted with the probe in air or distilled water.

Dissolved oxygen. Calibration of the sensor is accomplished by setting two points; one at zero with the sensor immersed in a saturated sodium sulphite solution, and a second equivalent to the measured dissolved oxygen of a stream sample as deter-

mined by the Altsterberg-Azide modification of the Winkler procedure as described in *Standard Methods.*

pH. The pH measuring system is calibrated using three standard buffer solutions, low, high and mid-range. The pH value of the buffer solutions used would depend on the range of pH values usually encountered.

Turbidity. Calibration of the submersible nephelometer sensor is accomplished by immersing the sensor in three different solutions of formazin (hydrazine sulphate and hexamethylenetetramine).

With this preventative maintenance and servicing schedule we are able to obtain from 60 to 70 per cent usable record. However, this is a costly operation since one operator can only maintain approximately three stations.

DATA PROCESSING

Automatic data processing is fundamental to the efficient operation of a nation-wide monitoring network. The automatic processing system used by the Geological Survey is a multipurpose system designed to translate, process, and store hydrologic data.

Water-quality parameters sensed by the electronic monitor are punched on 16-channel paper tape at the site. The paper tapes are submitted to the Data Processing Unit for processing, where they are examined to determine if the record length is correct and if there are any improperly spaced feed holes or torn tape. Translation of the 16-channel paper tape containing from one to ten water-quality parameters is done by a special-purpose off-line magnetic tape translator which records data on an IBM-compatible 7-channel magnetic tape at a recording density of 200 bits per inch.

Processing is relatively simple. The dial reading from the digital recorder and the engineering units at the end of straight line prorations are the only values required by the computer programme because the conversions are linear relationships.

A "Test Difference" evaluation is performed as an automatic edit to detect gross errors in the record. If the absolute difference between successive punch readings exceeds a designated test difference, an identifying symbol is printed out at the beginning of the summary for that day. Following this test, the primary computation is completed and a summary of the results, consisting of daily maximum, minimum and mean values for each item computed, is added to the Department of Interior "Storet" data file. A printout of this summary plus a listing of bihourly readings for each parameter is produced. In addition, a plot is prepared which contains the maximum, minimum, and mean for each parameter on the 16-channel tape, utilizing a cathode ray tube plotter. These machine plots are helpful to field offices in locating equipment, probe and/or pump failures and assist in estimating values for anomalous periods.

CONCLUSION

Automated water-quality monitoring has not resulted in the utopia of which many people first dreamed. We do not consider the level of approximately 60 to 70 per cent usable record now being collected as satisfactory, especially in light of the amount of effort expended. However, the reliability of the sensors and associated

instrumentation is gradually being improved and we are developing more skill and better techniques in operating them. While it does not appear that any dramatic breakthrough is on the immediate horizon, enough progress has and is being made to justify continuing in our efforts in this area.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Smoot, G. F.; Blakey, J. F. 1966. Systems for monitoring and digitally recording water-quality parameters. *U.S. Geol. Survey Open-File Report,* 14 p.

Standard methods for the examination of water and wastewater, 12th Edition, 1967. American Public Health Assoc., Inc., 769 p.

McCartney, D.; Beamer, N. H. 1965. Instrumentation in stream quality studies. *Industrial Water Engineering,* Feb., p. 10-13.

Radio control of water level gauges in watercourses endangered by high water levels

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Abstract. In regulated watercourses of the type operated for decades by Emschergonossenschaft and Lippeverband, high water levels can develop within a very short amount of time and with only a very little time lag after rainfall, especially in small catchments covering areas of only a few square kilometres. In order to avert damage and danger, it is indispensable to have information as early as possible on the different water levels at important points. Numerous flow measuring posts established by the two associations are susceptible to defects and disturbances, are slow in operation, and are rather expensive to operate and maintain. Therefore, Emschergenossenschaft and Lippeverband developed, in co-operation with AEG Telefunken, a system for the radio transmission of water levels to a central control station, in which the above disadvantages are avoided. Tests have shown that this teletransmission system has a notable recording accuracy, and that a tele-recording system will, in principle, be possible. The costs of such a system are lower than those for telephone transmission of water levels. Several of the new radio transmission flow measuring posts started operation in 1970.

TRANSMISSION PAR RADIO DES HAUTEURS LIMNIMETRIQUES DES COURS D'EAU PRESENTANT UN DANGER DE CRUE

Résumé. Sur des cours d'eau aménagés tels que ceux sur lesquels travaillent l'Emschergenossenschaft et le Lippeverband, des crues peuvent se produire en l'espace d'un quart d'heure et très peu de temps après la chute de pluie, et ce tout particulièrement sur de petits bassins versants de quelques kilomètres carrés.

Afin de limiter les dégâts et les risques, il est nécessaire d'être informé le plus rapidement possible sur le niveau des eaux aux points importants.

De nombreux postes de mesure automatique des hauteurs limnimétriques installés par les organismes compétents ont un fonctionnement lent et défectueux ; en outre, les frais d'exploitation et d'entretien sont importants. C'est pourquoi les deux organismes susmentionnés ont installé – en coopération avec AEG-Telefunken – un système radio de transmission des niveaux d'eau à un poste central, système qui ne présente pas les inconvénients précités.

La précision des indications obtenues lors des essais est bonne et, en principe, un téléenregistrement graphique est possible. Les frais sont plus faibles que pour la transmission par téléphone. L'exploitation de plusieurs installations semblables a commencé en 1970.

CONTROL SIN HILOS DE MEDIDORES DE NIVEL DE AGUA EN CURSOS AMERAZADOS POR LAS CRECIDAS

Resumen. En los cursos de agua regulados del tipo que durante decenios han trabajado la Emschergenossenschaft y la Lippeverband, las crecidas se pueden producir, especialmente en zonas pequeñas torrenciales que cubren nada más una zona de unos pocos km^2, en pocos minutos y con sólo un pequeño intervalo de tiempo después de las respectivas precipitaciones. A fin de advertir los daños y peligros, es indispensable una infromación rápida de los diferentes niveles de agua en puntos importantes. Ambas asociaciones han establecido numerosos puestos de medición de caudal, que pueden tener defectos y perturbaciones, son de funcionamiento "lento" y bastante caros de explotación y mantenimiento.

Por tanto, la Emschergenossenschaft y la Lippeverband pusieron a punto, en cooperación con AEG Telefunken, un sistema de transmisión sin hilos de los niveles de agua a la estación

central de control evitándose de esta forma las desventajas indicadas. Según indicaron los ensayos, esta teletransmisión posee una notable precisión de registro, y es posible el sistema de telerregistro. Los costos de dicho sistema son inferiores a los de transmisión telefónica de niveles de agua. Algunos de los nuevos sistemas de puestos de medición con transmisión se pusieron en servicio en 1970.

БЕПРОВОЛОЧНЫЙ КОНТРОЛЬ ЗА ИЗМЕРЕНИЯМИ УРОВНЯ ВОДЫ В ВОДОТОКАХ, ПОДВЕРЖЕННЫХ ПАВОДКАМ

Резюме. В регулируемых водотоках типа таких, которые эксплуатируются в течение нескольких десятилетий водохозяйственными управлениями рек Эмшер и Липпе, паводок может развиться в течение очень короткого периода времени и почти сразу же после соответствующего ливня, особенно на малых водосборах площадью в несколько квадратных километров. Для предотвращения раздушений и опасности необходимо как можно раньше получать информацию о разных уровнях воды в важных пунктах. Многочисленные посты для измерения стока, установленные двумя упомянутыми управлениями, несовершенны и подвержены неисправностям, требуют много времени для записи данных и дороги в управлении и обслуживании.

Поэтому управления рек Эмшер и Липпе, совместно с компанией Телефуикен, разработали систему беспроволочной передачи сведений об уровнях воды на центральную контрольную станцию; с помощью этой системы преодолены вышеупомянутые недостатки. Испытания показали, что эта система телепередачи обеспечивает сравнительно высокую точность записи и что телерегистрирующая система в принципе возможна. Стоимость такой системы ниже, чем стоимость системы телефонной передачи сведений об уровнях воды. Несколько новых постов для измерения стока, оборудованных беспроволочной системой передачи, вступили в строй в 1970 г.

For several decades Emschergenossenschaft and Lippeverband have operated a system of watercourses approximately 500 km long in the densely populated industrial district of Nordrhein-Westfalen. This system serves for water drainage from catchment areas varying in size from several hundred hectares to several hundred square kilometres (Fig. 1). The associations deal also with the River Lippe which, at its mouth, has a catchment area of 4,800 km^2. Lippeverband is responsible for flood protection in its area.

One of the primary difficulties of these two Associations responsible for the safety of the Ruhr District from a water engineering aspect is that large areas (in the environs of the River Emscher, one-third of the total area) are "polders" which were formed by subsidence of the surface due to mining operations. These polders must be drained by pumping stations. The sections of brooks, operated by the Associations, are provided with dykes up to 10 m high. The protection against high water levels of the dykes themselves, as well as for the population endangered in the case of a catastrophe, necessitates rapid and reliable control of discharges. Water level gauges with automatic data transmission are most important for this work.

Flood protection and the rapid reporting of high water levels are rendered difficult by their frequently very rapid onset after heavy rainfall. This occurs largely because of the reduction in rainfall infiltration consequent upon urbanization, which is still increasing, and is also aggravated by sewage and drainage systems necessitated by the urban development. Hence heavy rainfalls are quickly transformed into high discharge volumes (Fig. 2). The watercourses operated by the Associations were designed in the form of canals and do not diminish the trend for steeply increasing

discharge hydrographs. Thus little time is left for supply of water level information for analysis and for a warning service.

The Associations started to install automatically operated water level gauges ten years ago; since then, five such gauges have been established. Operational defects, however, occurred occasionally; thus at the time of the Lippe high-water in February 1970, the automatic reports from three of the five water level gauges along the river were defective or otherwise useless. Apart from defects in the mechanism of the water level gauges, the following were the sources of defect: (a) no telephone connexion: and (b) incomplete pick-up of tones from the record (non-functioning of one digit of the water level figure).

One cause was that the water level gauges were dialled not only by officials, but also by private people, newspapers and interested industrial enterprises, with the result that a repeated dialling was often necessary in order to receive the required information. To keep the telephone number secret is hardly practicable.

A feasible solution was found by utilizing and enlarging the radio network established by the Associations in 1968 for the supply of information during high-water. This network is operated by ultra-short wave transmitters and receivers at frequencies of 149.17 MHz and 149.31 MHz. These were so-called "exclusive frequencies", which may be used by not more than one other party within the "Rhombic Wave Distribution Plan" drawn up by the Postal Authorities for each region of the Federal Republic of Germany.

The two Associations now operate six water level gauges with stationary transmitters and high aerials as well as from four to six portable devices, type Teleport VI AEG Telefunken, for each water level gauge. At a sending range of between 5 and 15 km, according to circumstances, these transmitters cover the entire catchment area. Transmitters with a sending power of 1.5 Watt for an operating time of 10 hr, have proved to be reliable for the supply of information from field personnel to the central control station in numerous cases of high water levels.

This radio network could be used for building up a central system for the radio transmission of the water levels from important gauges to a central station. Transmitter and receiver were given the character of movable radio connexions; in other words, portable transmitters and receivers were installed in such a way that they could be used, at any time, for communicating information between moving points.

Because water level gauges are always situated in valleys, they are difficult to reach by radio, particularly using ultra-short wave equipment, as the range of such transmitters is a quasi-optical one. Hence long aerials had to be provided.

The set-up of the system is as simple as possible (Fig. 3) and is a follows.

1. Measuring of the water level by a gauge in a housing of simple design, consisting of concrete rings of diameter 1 m, a steel cover and a steel sheet casing. The casing accommodates at Ott-type water level recorder and parts of the radio equipment. As a protection against corrosion, it is advisable to heat the casing.
2. Transformation of the mechanical indication into a proportional electrical d.c. voltage by means of a helical potentiometer coupled to the axis of the recording device. The range of variation of the water level between the minimum water level and the highest water level will be up to 8 m. The potentiometer, however, has an invariable total range of 0 to 100 per cent or 360°. For as precise an indication as possible, the range of water levels had thus to be restricted to the upper 4 m.
3. When called from the central control station, the radio water level gauges called by a selective signal transmit the value of their stage via a Teleport device. For the duration of this call, i.e. 20 sec, the other radio telephony must be stopped. The

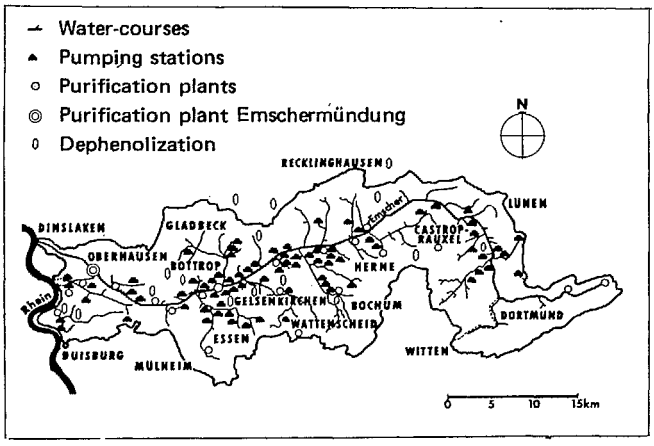


Figure 1 – *Catchment area and purification plants of the Emschergenossenschaft*

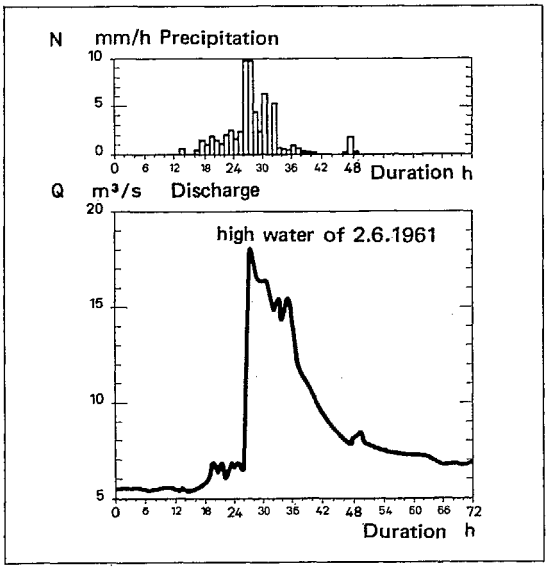


Figure 2 – *Hydrograph of rainfall and discharge*

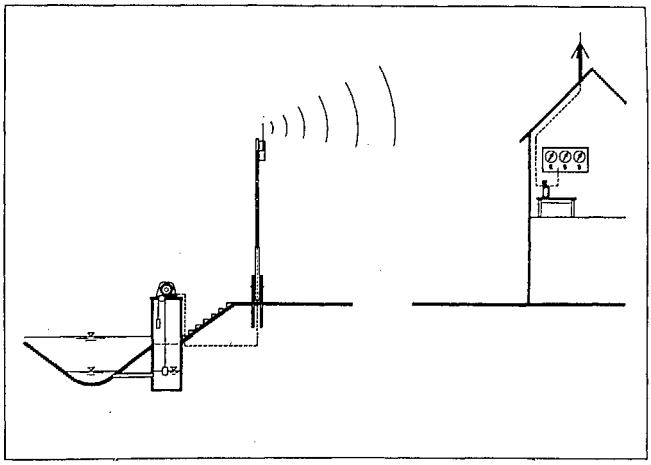


Figure 3 – *Set-up for the radio control system of water level gauges*

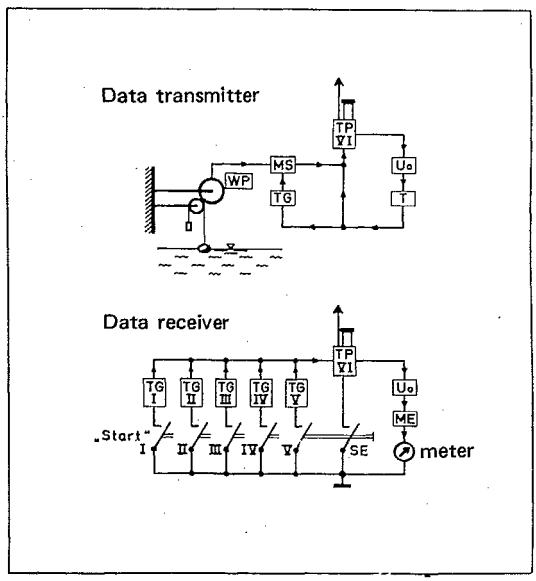


Figure 4 – *The switching system for radio control of water level gauges*

data transmitter is attached to the upper end of the high aerial. Power is supplied from the ordinary 220 volt grid.

4. The data from each water level gauge are shown separately in the central control station. After calibration, the indication can be shown directly as a water level. The water level can also be recorded using a dotted line recorder.

The switching system is as follows (Fig. 4): to initiate an enquiry, the transmitter in the central control station is switched on by depressing a key (SE). The sound generator (TG), switched on at the same time, modulates its audio-frequency call in the transmitter receiver. This modulated high-frequency signal is transmitted at first to all trnasmitters/receivers at the water level gauges. There, the general call is selectively analysed by the one-sound-re-transmitter (Uo), so as to call only the wanted gauging station. Then the following functions are actuated. Conversion of the value from the water level gauge, via the helical potentiometer (WP), at first into a proportional d.c. voltage, and then, by the transmitter supplying the measured value (MS), into a proportional a.c. voltage from 0 to 12 Hz. This frequency is then used for its own call (TP). The twice modulated high-frequency is transmitted for 20 sec set in the time element (T) and is received in the central control station. The high-frequency signal received is demodulated, the one-sound-retransmitter (Uo) responds and the receiver of the measured value (ME) gets the frequency of 0 to 12 Hz. This again is formed into a proportional d.c. voltage which is displayed.

The time element terminates the calling procedure and all devices are switched back into their neutral position.

The multiple conversion of the originally mechanical indication into electric d.c. and a.c. voltages and into audio-frequencies gave rise to the suspicion that other considerable errors in transmission might be expected. In order to be safe, a prototype with the structural elements of the planned system was established and tried in the field, prior to placing the definite order with the supply firm.

The signals received showed an almost faultless agreement with the water levels transmitted by radio-telephony at the same time. In order to be quite sure, several rising and falling water stages were simulated and transmitted, but again there were no differences. The precision of transmission amounted to ± 2 cm water level at a total range of 9 m. The final equipment with its 4 m range of measurement will have even better precision.

In conclusion, the cost of a radio water level gauging station must be compared with those of the previous gauging stations with automatic transmission by telephone. The following figures represent the average prices of the 4 or 5 installations established so far.

1. *Water level gauges with automatic telephone:*

Building for water level recorder, underground building, measuring installations, equipment for automatic answering of calls, telephone

Building cost	37,000 DM
Equipment	12,000 DM
Total	49,000 DM

2. *Water level gauges with radio transmission of data:*

Gauging station of ordinary design, measuring installations, complete set of transmitters and receivers

Building cost	3,500 DM
Equipment	4,950 DM

Share of central control station for 5 transmitters/receivers	1,050 DM
Other cost (mast for high aerial, power supply, mounting)	700 DM
Total	10,200 DM

Thus the cost for the new gauging stations with radio transmission amounted to only about one-fifth of that of the previous gauging stations with automatic telephoning of information. Better protection against dust and moisture must be provided for maintenance of the water level gauging stations with the telephone service, as these gauges have mechanically moved parts, whereas the radio ones need only a moving potentiometer. The cost was also lower for the radio gauging station because of the use of pre-fabricated structural elements. As these radio water level gauges quite fulfil their duties they can be considered fully equivalent to the water level gauges with telephonic service. There is not yet much experience of the operating costs and the operating safety of the new system.

DISCUSSION

Mundt: Our teletransmission of gauges at present uses the network of the Post Office in a rather complicated way. Is it possible to install a radio station and then to use a less complicated way, particularly in cases of longer distances?

Oebius: In Wilhelmshaven we have had the same difficulty when using the telephone line. In principle it is possible to transfer data by using a telephone line, not on an acoustic basis, but with the help of modulated frequencies. The problem, however, concerns the distances.

Did you contact a producer for reducing the deviation of ± 2 cm? I believe that it should be possible to combine two potentiometers in a way that the one measures the interval from 0 to 8 m and the other measures from 0 to 1 m, but operating on the first one. This would reduce the error to ± 1 cm.

Stalmann: Our device normally offers the necessary accuracy, but I shall consider your idea. But I should like to report on another difficulty. During the last flood we also had a storm, and when there were flashes of lighting, we could not operate the instrument.

De Backer: I should like to mention that this morning we discussed the transmission from different water gaugings along the river. We developed the system in Belgium at the Laboratory of Agricultural Engineering and we are using the public telephone network for transmission. The cost of each water gauge station is much lower than the one you presented. We do not necessarily use equipment with a very high precision. We use different systems. We make the calibration of each measurement so that we do not depend on the drift due to temperature, humidity and the use of electronic components. We are using a potentiometer as a means of translating the water levels.

A computer automated system for hydrologic data acquisition and analyses

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Abstract. An interfaced hydrologic data acquisition-processing system has been installed at the University of Arizona. A forested mountain watershed and a desert watershed area within a 50 km radius of the campus are instrumented for measuring a variety of hydrologic and climatic variables. Measurement data are both multiplexed over the Bell Telephone system and transmitted by radio to a campus laboratory. Measurements on the desert watershed may be programmed in any sequence and sampling rate varied from seconds to 24-hr intervals automatically or according to the rate at which input signals may change. Operation of the system is automated from a central laboratory by a small computer. Reference information, preliminary data reduction programmes, and control data may be entered into the system. All output is recorded on magnetic tape, but may also be displayed on strip charts or teleprinter. The facility is used in an ecological research programme and provides input-output data for research in synthesizing the hydrologic behaviour of watersheds. It also offers a unique means of teaching principles of hydrologic processes, methods of data acquisition and procedures in hydrologic analyses.

SYSTEME AUTOMATIQUE POUR LA COLLECTE ET L'ANALYSE DES DONNEES HYDROLOGIQUES PAR ORDINATEUR

Résumé. Un système intégré de collecte et de traitement des informations hydrologiques a été installé à l'Université de l'Arizona. Deux bassins versants sont équipés pour mesurer une grande diversité de paramètres hydrologiques et climatologiques : un bassin forestier et un bassin désertique. Ils se trouvent à 50 kilomètres de l'université. Les deux types d'information passent simultanément (méthode "multiplex") par le système de communication de la compagnie de téléphone Bell, d'où elles sont transmises par radio au laboratoire de l'université. Les mesures sur le bassin désertique peuvent être effectuées à une cadence quelconque (dont la fréquence peut varier de quelques secondes à 24 heures), automatiquement ou selon la fréquence des signaux envoyés. Le fonctionnement du système est automatiquement contrôlé à partir du laboratoire central au moyen d'un ordinateur. Le système peut être alimenté par des informations de référence, des programmes de réduction d'informations préliminaires, et des informations de contrôle. Le résultat est normalement enregistré sur bande magnétique, mais il peut aussi être enregistré sur papier millimétré ou par téléimprimeur. Cette installation est utilisée dans un programme de recherches écologiques pour lequel elle fournit les informations nécessaires aux recherches de synthèse hydrologique sur les bassins versants. Elle constitue aussi un moyen unique d'enseigner les principes des processus hydrologiques, les méthodes de collecte des données et les méthodes d'analyse hydrologique.

SISTEMA AUTOMATIZADO CON ORDENADOR PARA RECOGIDA Y ANÁLISIS DE DATOS HIDROLÓGIGOS

Resumen. En la Universidad de Arizona se ha instalado un sistema de recogida y proceso de datos hidrológicos interfaciales. Se instalaron instrumentos en una cuenca de montaña con bosques y en otra desértica, en un radio de unos 50 km del campus para la medición de diversas variables hidrológicas y climatológicas. Los datos de medida se transmitieron por cable multiplex del sistema de la Bell Telephone y también por radio al laboratorio universitario. Las mediciones en la cuenca desértica se pueden programar en cualquier secuencia, y se hicieron tomas de muestras a

intervalos de segundos a 24 horas, automáticamente o según los cambios que se puedan hacer en las señales de entrada. El funcionamiento del sistema se automatiza desde un laboratorio central por medio de un pequeño ordenador. Se pueden introducir en el sistema información de referencia, programas de reducción de datos preliminares, y datos de control. Toda la salida se registra en cinta magnética, pero también se puede obtener en gráficos de cinta o en teleimpresor. La instalación se usa en un programa de investigación ecológica y da datos de entrada-salida para investigación de síntesis de comportamiento hidrológico de las cuencas. También constituye un medio único para enseñar principios de procesos hidrológicos, métodos de adquisición de datos y procedimientos para análisis de datos hidrológicos.

СЧЕТНАЯ АВТОМАТИЧЕСКАЯ СИСТЕМА ДЛЯ ПОЛУЧЕНИЯ И АНАЛИЗА ГИДРОЛОГИЧЕСКИХ ДАННЫХ

Резюме. Система дляполучения и обработки гидрологических данных была установлена в Университете штата Аризона. Площади водосборов : лесистого горного и пустынного, в радиусе 50 км от университетского городка, оборудованы для измерений разнообразных гидрологических и климатических переменных. Данные измерений могут передаваться как с помощью телефонной связи, так и по радио в университетскую лабораторию. Измерения на пустынном водосборе могут быть запрограммированы в любой последовательности, и частота измерений изменялась от секундных до 24-х часовых интервалов автоматически или в соответствии с интенсивностью, с которой могут изменяться вводимые сигналы. Управление системой производится автоматически из лаборатории с помощью небольшого компьютера. В систему может быть введена справочная инфорнация, предварительные программы обработки данных и контрольные данные. Весь выход данных регистрируется на магинитной ленте, но также может быть записан на ленточных диаграммах и на телетайпе. Это оборудование используется в экологических исследовательских программах и дает информацию входа — выхода для исследования при обобщении гидрологического режима водосборов. Оно также дает уникальные средства преподавания принципов гидрологических процессов, методов получения данных и приемов гидрологического анализа.

INTRODUCTION

Deserts of the United States support one of the most rapidly increasing population segments of the nation. The associated sprawl of compacted urban areas and the changing use patterns of wildlands will emphasize environmental pressures on the "fragile" desert ecology in the future, particularly when more water becomes available through technological advances in desalinization, weather modification and watershed management practices.

To sustain desirable aspects of this growth a thorough knowledge of the hydrological and ecological processes characterizing deserts is essential. Land managers must have a wider understanding and more technical training than ever before. Research results must be synthesized into predictive models to serve as guides for optimizing resource management and urban planning strategies.

This paper describes a data acquisition facility being developed for hydrological and ecological research and training, operated jointly by the College of Earth Sciences and the Department of Watershed Management, in the College of Agriculture, at the University of Arizona.

THE FACILITY

The complete facility will include a forested mountain watershed, a desert grassland watershed and a Sonoran desert watershed (Figure 1). At present, two modes of telemetry are used to transmit a variety of hydrological and meteorological data from the forest and desert grassland watersheds to a central control station on campus. The Sonoran desert watershed will be added to the facility with a hard wire telemetry link carly in 1971.

The central station contains a small computer for on-line data processing and recording. It affords control of the data taking process—varying the sample rate in relation to input changes, real time, etc. Magnetic tape permanently records the data.

Three major functions of the facility provide: (1) meteorological-hydrological input for an ecological research effort of the Diesert Biome[1]; (2) quality control in

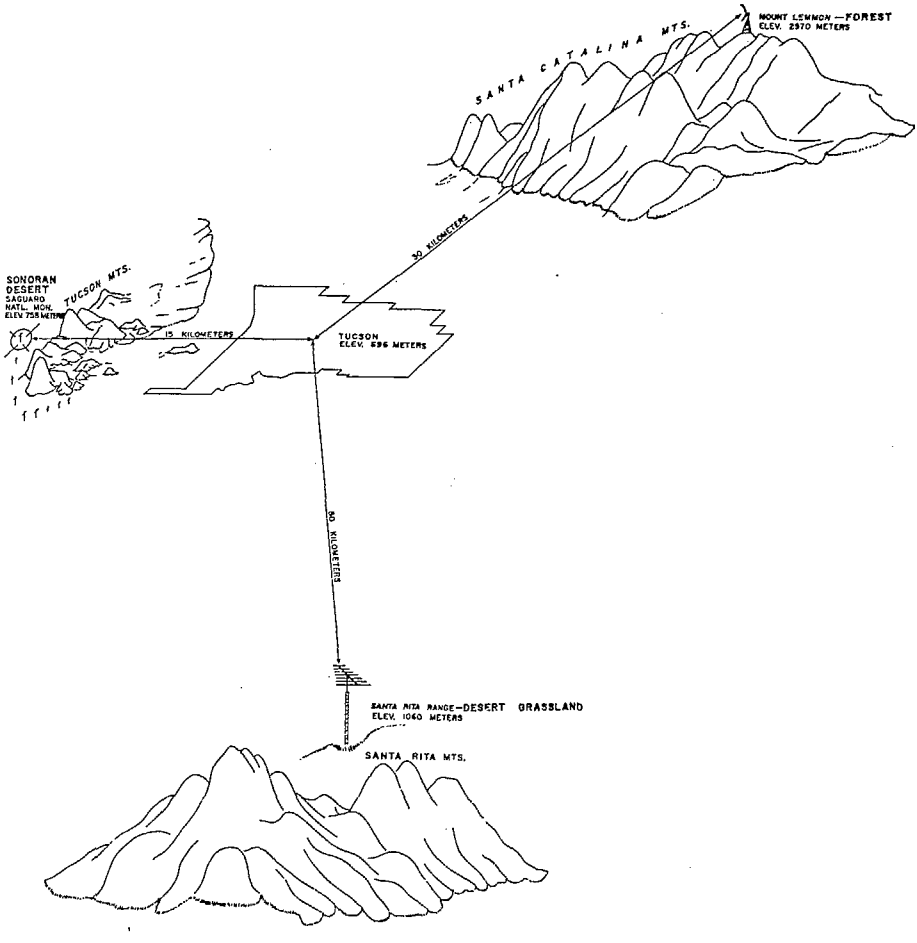


Figure 1 – *Location of the facility*

1. International Biological Program, Research Design Plan for Analysis of Structive and Function of Desert Ecosystems, David W. Goodall, Director, Utah State University, 1969.

the collection of input-output data needed for developing and testing synthetic hydrologic models of watersheds; and (3) a unique laboratory for student instruction in watershed hydrology.

TELEMETRY AND DATA ACQUISITION SYSTEMS

The facility tests hydrologic data acquisition schemes and provides a variety of instrumentation for student instruction. One of the two systems in operation is a simple impulse duration type. Data are transmitted by tone pulses of durations proportional to variations in the measured qantities and are converted at the central station into analogue form. The other is a state of the art, remote control digital data acquisition system.

PULSE DURATION SYSTEM

An electro-mechanical transmitter, the principal element of this system, converts variations in a measured quantity to a proportional time interval (Fig. 2). The conversion is made by a follower which rides a continuously rotating cam and keys a mercury switch. The position of the follower on the cam, set by an electrical or mechanical link to the sensor, determines the duration of the pulse. A 13 sec pulse corresponds to maximum deflection of a sensor, and a 3 sec pulse to zero. Thus, measurement data are carried within a 3 to 13 sec time interval for cach 15 sec cam cycle.

Time modulated pulses from the transmitters key corresponding tone oscillators. The resulting modulated tones are multiplexed through an underground telephone cable to a microwave transmitter on the watershed summit. Multiplex data are transmitted to the Tucson telephone exchange and are sent via a leased line to the central station. The system has a capacity of 24 channels imposed by the audio frequency spectrum of the commercial telephone facility. Individual channels are separated by 100 Hz.

At the central station, the signal is demodulated to direct current pulses by bandpass filters and tone receivers. Time durations of pulses correspond to amplitudes of the original sensor output. Direct current pulses position retransmitting slide wires and, for selected variables, operate analogue recorders for visual monitoring. The retransmitting slide wires are read by a magnetic tape data logger. But the system is also being interfaced with the control station of the digital system.

DIGITAL SYSTEM

This system consists at present of a control station at Tucson and a remote station at the Santa Rita desert grassland site. Both digital pulse and analogue type sensors are used at the remote station (Fig. 3). The digital pulse sensors are typical contact closure devices, such as a tipping bucket rain gauge. Signals from analogue sensors are conditioned and multiplexed into a 10-bit, ramp type, analogue to digital converter. The digitized output of the converter is applied via the data bus into the memory.

The memory is a 32-word, 16-bit core plane with control logic. A pulse from either type of sensor enters into the memory via the data bus. The basic function of the memory permits digital pulse data to be stored between interrogation cycles. At

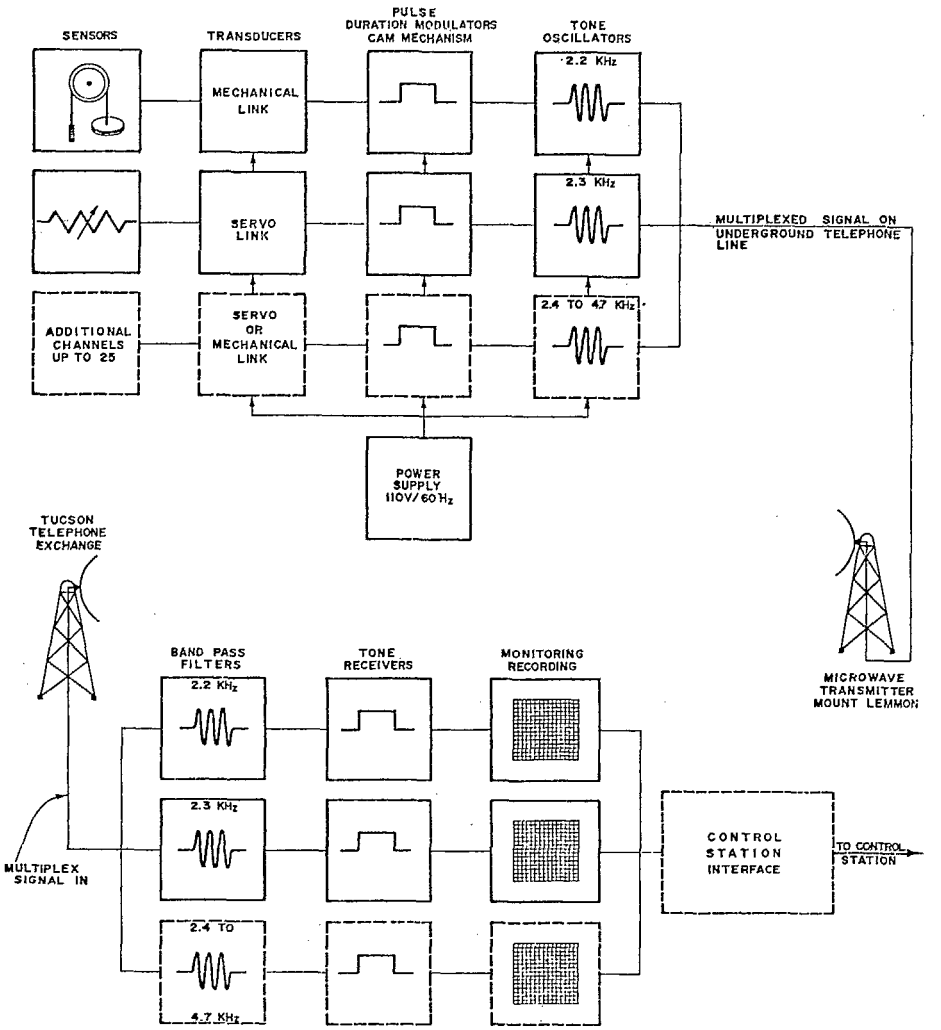


Figure 2 – *Pulse duration system*

the beginning of an interrogation cycle, the analogue data are written into memory and a sequential output of all data begun.

As data are being sequenced out of memory a parity bit is assigned. Parity checking determines if data have been correctly transmitted and received. The number of bits in a data word are totalled, and if an even number of bits is present, then a parity bit is added to make the total odd. If an error occurs in the transmission of data (as might happen during an electrical storm), the error is detected and the data flagged.

Interrogation of the remote stations is initiated by a signal generated at the control station and received by the tone decoder which then initiates a sampling and reporting cycle. Output from the memory, applied to the tone encoder, modulates the transmitter carrier for data transfer to the control station.

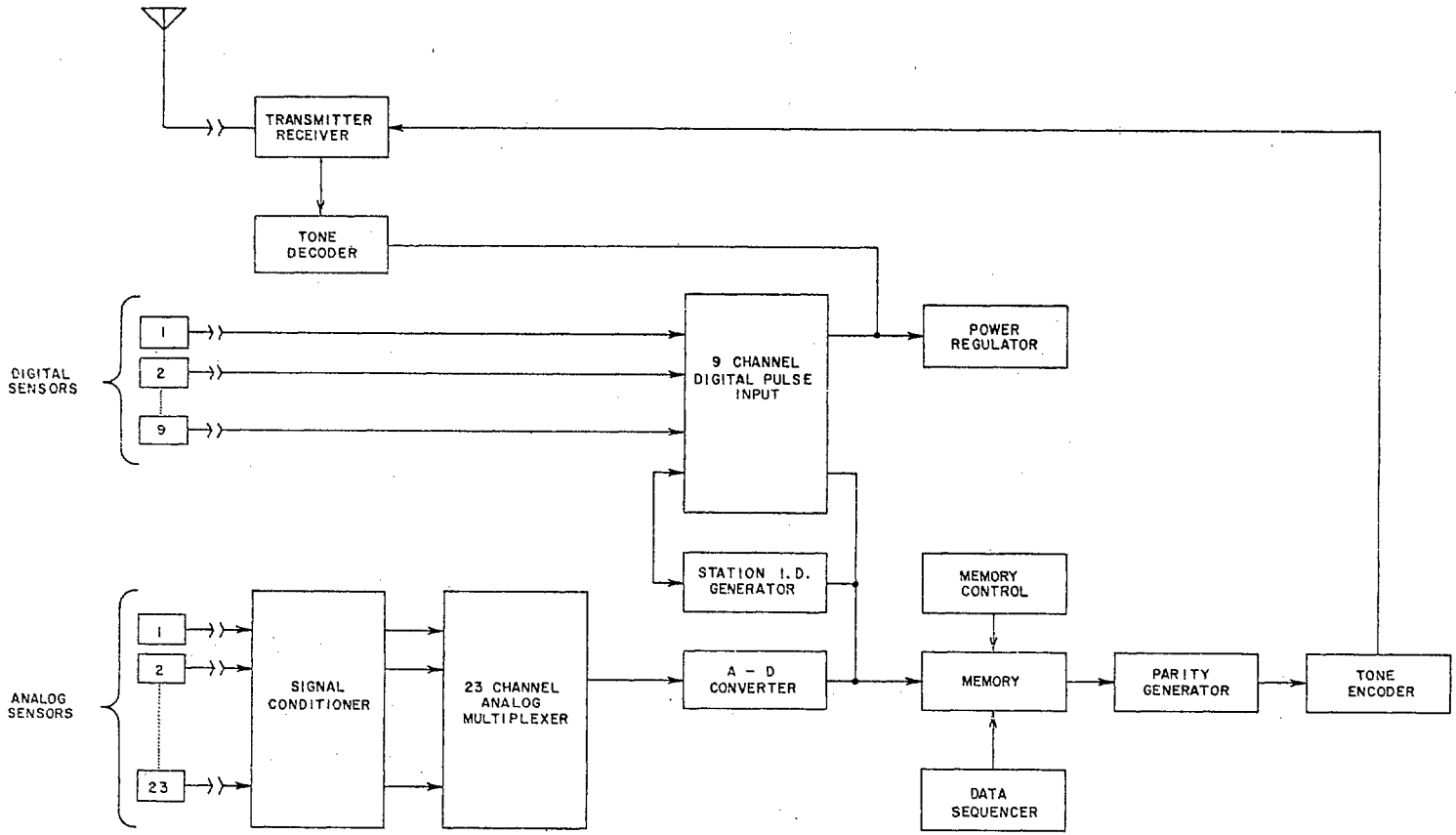


Figure 3 – *Digital system, remote station*

The control station generates remote station addresses, checks them as well as incoming data for validity, and logs all information. The equipment consists of a teleprinter and an equipment rack in which are mounted (1) crystal controlled digital clock, (2) radio communication link transceiver, (3) read/write magnetic tape deck, (4) 8K 16-bit computer, and (5) high speed paper tape reader.

The clock displays real time in hours, minutes and seconds and is monitored by the computer. When a selected time interval has elapsed, pulses are generated by the clock and an interrogation cycle is begun. The time interval between interrogation cycles is adjustable by operator entry either directly through the teleprinter keyboard or through the tape recorder with paper tape. A code may also be entered manually into the teleprinter to initiate a manual request for data from either all stations in sequence or from a selected station.

An interrogation cycle begins with the computer outputting to the transceiver an 8-bit address word, the addressed remote station responds and transmits the accumulated data back to the control station, the received data is checked for the correct station address, correct number of bits in the record and correct parity. If these are positive, the data are then converted into engineering terms before they are logged, e.g. wind direction in degrees, wind velocity in miles per hour, and temperature in degrees, etc. The control station then interrogates the next station. Read out is normally made every six minutes and transcribed on magnetic tape. The teleprinter displays the data every hour except during storms when the sampling and display frequency is stepped up to two minutes.

Magnetic tape at the control station provides the optimum raw data storage medium for the facility. It records all data as they are acquired and recalls historic data at will for new or additional analyses without restricting system capability. The small computer performs many of the analyses required by the project. For complex problems, such as those associated with computer modelling, the tape is used directly on the 6 400 CDC computer.

COMPARISON OF THE TWO SYSTEMS

The pulse-duration system has been in operation for two years, the digital system for only a few months. For this reason and because of differences in their basic function, direct comparison cannot be made.

Essentially, the pulse-duration system is a telemetry system for transmitting data from the field and recording it in the laboratory. Expensive interfacing with the central computer is being improvised to make it a more complete data acquisition system. Even so, it will lack the flexibility of the digital system.

Interestingly, costs of field stations and telemetry links are similar for both systems. The field station components of the digital system are contained in compact, light-weight, water-tight units which may be buried underground. Electronics are solid state for reliability and have low power consumption. The system, normally in a stand-by condition, consumes little power because it is activated only when it receives an interrogation command.

The pulse duration field installation operates continuously and requires 110V/60 Hz power which presents a serious problem for isolated instruments. Because the system is used near a mountain summit, above ground power lines are impractical for winter conditions, and the buried lines in use pick up induced power surges whenever lightning strikes the watershed. This problem has not been overcome.

Field transmitters of the pulse-duration system, though ruggedly constructed, are exposed above ground. The mechanical mechanism wears under continuous use and requires excessive maintenance; however, repairs are not difficult to make. The solid state components of the digital system have thus far been trouble free, but in case of breakdown, the services of a skilled technician are required.

ECOLOGICAL FUNCTION

A major goal of the Desert Biome programme is to develop predictive models of the transfer of energy and matter among the components of desert ecosystems. A large number of desert sites in the western United States are involved in the over-all programme. The desert grassland and Sonoran desert are the primary sites in Arizona.

Knowledge of the energy and water balance is essential to the programme. Accordingly, the Santa Rita site is instrumented as shown in Table 1. A similar installation will replace a standard weather station at the Saguaro site in 1971. A less sophisticated set of instruments has been in operation on the forested watershed for two years.

Table 1. *Meteorological and soil measurements, desert grassland site*

Distance from soil surface	Measurement
+ 16	Water vapour, temperature, wind speed
+ 8	Water vapour, temperature, wind speed, wind direction
+ 4	Water vapour, temperature, wind speed, radiation
+ 1	Precipitation (one recording gauge per 100 acres)
0	Soil surface temperature (radiometer)
−0.05	Soil moisture, soil temperature, soil water potential, spatially replicated.
−0.10	
−0.25	
−0.50	
−1.00	
−2.00	

To estimate the energy balance, meteorological variables are continuously measured above the vegetation canopy at a single station on the study area. Supplementary measurements are obtained for short periods at other locations over the areas. Hopefully, these measurements augmented with data on the surface roughness of the vegetation can be extrapolated downwards into the vegetation layer.

The meteorological data required are: (a) profiles of wind, temperature and moisture, and (b) stress, roughness and heat flux in the surface boundary layer. Because there are alternative approaches to obtaining these quantities, as well as preferences by individual investigators for specific sensing instruments, both digital and analogue, among the wide array available, an extremely adaptable data acquisition system is required.

The water balance on the ecological sites is being assessed by two methods; direct measurement and hydrologic simulation. Direct measurements are made of major inputs and outputs including precipitation, stream flow, channel transmission loss and soil moisture.

Spatial estimates of precipitation in the southern desert of the United States are made difficult by convection storms which characteristically occur in cells not much larger than 1 km in diameter. Tipping bucket raingauges, monitored by the digital system, and supplemented with standard gauges are proving adequate. The tipping bucket gauge is well suited to the project; it requires little service, minimizes disturbance on the study areas, and can be installed at considerable distance from the remote station with only a simple line connection.

Typical desert watersheds have low relief dry dense channel networks and intermittent stream flow. Channel transmission losses are high and difficult to evaluate. At present stream flow is being measured at inlet and outlet points on two channels on the desert grassland watershed by four flumes equipped with telemetry. The network will be expanded in the future.

Soil moisture, because of spatial variability is sampled directly. However, plans are in progress to test aerial infra-red scanning, supplemented with ground truth data on soil moisture and rainfall to obtain spatial estimates.

SIMULATION FUNCTION

Simulation modelling, one of the most popular tools of hydrology, produces a history of events based on mathematical models representing real world situations. Frequently the accuracy of available data is the limiting criterion and establishes the simulation accuracy of a particular model. Data requirements for simulation must be guided by the model building process. For maximum efficiency the entire process of data collection, evaluation, conversion and storage, through the final analysis should be a single system. In this way transmission of data from one stage to the next and the effect of changing one part of the system or another, can be kept in the forefront. The facility described here has this capability.

Because of the relatively high precipitation on desert mountains and because these areas make important contributions to the region, the forested mountain watershed is used primarily to provide input-output data for hydrologic modelling.

The hydrologic quantities measured and telemetered are surface runoff from plots replicated on the watershed, stream flow from the watershed, precipitation and snow density. Climatic variables monitored are solar radiation, humidity and air temperature.

INSTRUCTIONAL FUNCTION

Aside from its inherent research functions, the facility is a key part of a curricula being developed in watershed hydrology. Specific instructional uses give students: (1) fundamental understanding of the interrelationships between variables in the hydrologic cycle by working first-hand in the laboratory with these quantities as they change in space and time on natural watersheds; (2) technical training by performing laboratory exercises in computer programming for data acquisition, reduction and analysis.

By means of high speed computation data may be collected, reduced and analysed in any desired fashion almost immediately and in some cases while the hydrologic event is occurring. The student observes and works with hydrologic processes in the laboratory, at any time and under the most inclement conditions. With this versatility the gap between the publication of research results and their eventual incorpora-

tion. into classroom instruction is lessened. The convenience of having a variety of data immediately at hand and ready for the computer allows new developments in hydrologic techniques and research to be tested and demonstrated with the facility as they become pertinent. Teaching efficiency and educational effectiveness is increased by dealing with complex problems in a very practical but dramatic way.

The small computer provides an economical means of teaching the application of computer principles in hydrology, ranging from basics to the implementation of graduate research projects. Data reduction schemes, operational computer programmes and laboratory exercises are about 75 per cent complete at present. Because the facility is open-ended to allow additional or deletion of new developments, its full potential will probably not be realized for some time. It will be tested in several courses during the fall semester of 1970 and will go into full-time operation the following semester.

Acknowledgements. This project is partially supported by National Science Foundation Grand GB 15886 and by the Agency for International Development under an institutional grant to increase instructional competence in system analysis of watershed management.

DISCUSSION

Voermin: What are the costs of installation?

Thames: I estimate the costs at about $70,000, plus the cost of leasing equipment at quite a bit more.

Télécontrôle du niveau et du débit d'eau dans les systèmes d'irrigation

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Résumé. La télécommande de la répartition de l'eau dans les systèmes d'irrigation nécessite qu'à de grandes distances, on puisse mesurer d'une façon précise et efficace le niveau et le débit de l'eau en différents postes de contrôle ainsi que la position des vannes commandées.

L'auteur décrit un système de télémesure codifiée qui utilise des codeurs monodromes monovariables.

De tels codeurs, dont la construction est relativement simple et dont le prix est relativement bas, permettent de mesurer le niveau de l'eau dans une gamme de 1 à 4 mètres avec une précision de 1 centimètre.

Ce dispositif peut être couplé avec un système télémécanique du type "Temir" qui permet de contrôler à partir d'un centre unique jusqu'à 480 postes, de commander les vannes et de transmettre des signaux d'alarme lorsque les niveaux dépassent une cote donnée.

TELEMETRIC CONTROL OF WATER LEVEL AND DISCHARGE OF IRRIGATION SYSTEM

Abstract. The modern hydrometric service on irrigation systems requires a constant and operational control of water level and discharge at different points simultaneously, as well as measurements of the position of the control gates.

A code system for telemetric measurements of water level and discharge, as well as the position of the gates, operating over a radius of 80 km, has been elaborated in the Kirghiz Research Institute of Water Management. The sensor uses monodromic, monovariable code, providing a considerable decrease of its dimensions, and simplifying its manufacture. The coder is intended for 100 levels of quantization and is supplied with a supplementary unit, providing an extension of its range by 200 and 400 levels. An error of not more than ± 1 cm is tolerated in measurements of 1-4 m. The information is transmitted from the sensors to the central control panel, where an automatic decoding, digital indication and registration by a digital printing installation take place. A system of selective signals allows up to 48 sensors to use a pair of telephone wires. A single control panel may control up to 10 pairs of wires and thus up to 480 sensors. Besides the transmission of telemetric information, the installation transmits an alarm signal when a pre-set critical level is reached. The installation is also intended for the telemetric operation of the gates at hydraulic structures. The signal system and the sensors of telemetric measurements are fed from the control panel.

CONTROL TELEMÉTRICO DE NIVEL DE AGUA Y CAUDAL EN SISTEMAS DE RIEGO

Resumen. Un servicio hidrométrico moderno en sistemas de riego exige un control de nivel de agua y caudal de forma constante y funcional y en diversos puntos simultáneamente, así como mediciones de la posición de las compuertas de control.

En el Instituto Kirghiz de Investigación de Gestión de Aguas se ha elaborado un sistema de código de mediciones telemétricas de nivel de agua así como de la posición de las compuertas de control.

El detector utiliza un código monodrónico y monovariable, que produce una considerable disminución en sus dimensiones, acoplamiento axial y simplificación en su fabricación.

El codificador está proyectado para 100 niveles de cuantización y está alimentado por una unidad complementaria, que permite extender su amplitud en 200 y 400 niveles. Si la escala de lectura es de 1 cm, la amplitud de medición será de 1m, 2m y 4m.

La información se transmite de los detectores al panel de control central, en el cual tiene lugar una descodificación automática, una indicación digital y un registro por una instalación de impresión digital.

El sistema de señal selectiva permite la conexión de hasta 48 detectores a un acoplamiento general de los hilos telefónicos. El panel de control puede garantizar una conexión de hasta 10 pares de hilos y puede controlar hasta 480 estaciones utilizando un único panel central. Además de la transmisión de la información la instalación está preparada para transmitir señales de alarma cuando se alcanzan alturas críticas, y está preparada también para la operación telemétrica de las compuertas de control.

El sistema de señales y sensores de las medidas telemétricas se encuentra en el panel de control. El radio de operación del sistema es de 80 km.

ТЕЛЕКОНТРОЛЬ ГОРИЗОНТА И РАСХОДА ВОДЫ НА ИРРИГАЦИОННЫХ СИСТЕМАХ

Резюме. Современная служба гидрометрии на ирригационных системах требует постоянного оперативного контроля за уровнем и расходом одновременно на различных постах, а также измерения положения управляемых заворов.

В Киргизском научно-исследовательском институте водного хозяйства разработана кодовая система телеизмерения горизонта воды, расхода воды и положения затвора.

Датчик использует монодромный (однодорожковый) однопеременный код, что позволяет значительно снизить его габариты, момент на валу, а такжет упростить его изготовление.

Кодирующий преобразватель рассчитан на 100 уровней квантования и иметт приставки, позволяющие расширить его диапазон на 200 и 400 уровней.

При цене деления в 1 см, диапазон измерения будет 1 м, 2 м и 4 м.

Информация передается с датчиков на центральный диспетчерский пункт, где происходит автоматически расшифровка, цифровая индикация и регистрация при помощи цифропечатающего устройства.

Система селективного вызова позволяет подключить к одной общей паре телефонных проводов до 48 датчиков. К одному пульту управления может быть подключено до 10-и пар проводов, что доводит количество постов, контролируемых с единого центра, до 480. Кроме передачи телеметрической информации устройство передает с постов аварийный сигнал, когда отметка горизонта достигает критического значения. Устройство также расчитано на телеуправление затворами на гидротехнических сооружениях.

Питание аппаратуры вызова и датчиков телеизмерения осуществляется дистанционно с диспетчерсокого центра.

Радиус действия системы — 80 км.

L'exploitation des systèmes d'irrigation exige un contrôle et une répartition centralisés de l'eau. L'efficacité des mesures hydrométriques dépend même beaucoup de la précision et de la promptitude de la collecte des informations.

Une observation ininterrompue des phénomènes sur les systèmes d'irrigation et sur les cours d'eau et une répartition précise de l'eau permettent de réduire au minimum les pertes de celle-ci, d'augmenter la surface des terres irriguées à partir d'un canal au débit maximal déterminé et d'augmenter l'approvisionnement en eau des entreprises agricoles.

La commande automatique de n'importe quel système de production peut s'effectuer avec succès si l'on dispose de données sur l'installation commandée. C'est pour cette raison que sur les systèmes d'irrigation contemporains, on attribue une attention particulière aux moyens de contrôle et surtout de télécontrôle.

Lors de l'application des moyens de télécontrôle sur les systèmes d'irrigation, il faut tenir compte des principes suivants.

Les postes de mesure doivent être nombreux et répartis sur une large zone. Leur nombre peut varier, sur un système d'irrigation moyen, entre 40 et 100, et leur distance du centre de commande peut atteindre 60 à 70 km et quelquefois plus.

L'erreur tolérée lors de la télémesure des niveaux d'eau ne doit pas dépasser ± 1 cm pour des hauteurs allant de 1 à 4 m.

Les postes de mesure n'ont en général pas de source d'alimentation électrique.

Les conditions climatiques de fonctionnement des instruments de mesure sont aussi très dures. Ils doivent être conçus pour des variations de température allant de – 20 °C à 60 °C. Les instruments de mesure doivent être simples, fiables et leur prix doit être relativement bas.

Dans les systèmes d'irrigation, le débit est estimé, comme on le sait, à partir du niveau de l'eau ou en fonction de la différence de deux niveaux, là où il existe des ouvrages de jaugeage. Les instruments de télémesure doivent être conçus pour la mesure de ces grandeurs.

En outre, lors de la commande à distance des vannes, il faut évaluer la position de celles-ci.

Il faut enfin transmettre des signaux au centre lorsque le niveau atteint une cote limite (critique), afin de pouvoir, lors d'une crue ou de toute autre situation menaçante, intervenir rapidement et dans certains cas éviter la catastrophe.

A l'Institut kirghize de recherches scientifiques de l'eau, un dispositif a été élaboré permettant de mesurer à distance le niveau de l'eau, la différence de niveaux et la position d'une vanne, et qui répond aux principales exigences formulées ci-dessus.

Le dispositif utilise un codeur monodrome monovariable qui fournit une haute précision de mesure.

Un codeur monodrome pour n'importe quel rang (nombre de chiffres binaires) comporte une piste unique. Cela permet de simplifier d'une façon appréciable la construction des capteurs, de réduire les dimensions – et par conséquent le couple sur leur axe – du flotteur et du puits où se trouve celui-ci. Le diamètre extérieur du codeur est de 8 cm. Avec un flotteur de 10 cm de diamètre, il fournit une précision de 1 cm dans toute la gamme de mesure. Ce flotteur est placé dans un tuyau extérieur de faible diamètre qui se trouve placé directement dans le courant du canal, ce qui permet d'effectuer des mesures même dans de très petits canaux sans perturber leur courant.

L'utilisation d'un code monovariable pour lequel, dans toute la gamme de mesure, les signaux qui correspondent à deux points voisins diffèrent l'un de l'autre d'un seul élément permet de mesurer des grandeurs à variation monotone sans erreur de codage. En outre, l'utilisation d'un code monovariable permet de limiter les exigences en ce qui concerne la précision de fabrication et de montage du codeur.

Le codeur utilise des contacts de lecture. Une telle solution technique a été déterminée par les considérations suivantes.

La durée de vie des codeurs à contacts habituels est de 10^6 cycles complets. Etant donné que sur les systèmes d'irrigation les grandeurs mesurables varient lentement, de tels codeurs peuvent servir plus de dix ans.

Le couple sur l'axe, grâce à l'utilisation d'un code monodrome, est comparable au couple des capteurs sans contacts (environ 10 cm cN).

Les codeurs à contacts n'exigent pas d'amplificateurs intermédiaires et ils sont économiques du point de vue de l'alimentation électrique.

La transmission des signaux (pour 100 points) s'effectue par 5 impulsions d'un code tertiaire (possédant trois types d'impulsions) ou bien par 10 impulsions d'un

code binaire. Dans le premier cas, la durée de transmission sur une ligne téléphonique est de 0,5 seconde, dans le second cas de 1 seconde. Les impulsions sont transmises l'une après l'autre ; pour cela au poste de contrôle se trouve un commutateur commandé et alimenté à distance à partir du centre, par l'intermédiaire des mêmes fils qui servent à la transmission de l'information de retour.

Afin d'augmenter la gamme de mesure, le disque du capteur décrit plus haut peut accomplir plusieurs cycles ; après le passage au niveau "99", il retourne au niveau "00". Dans ce cas, un autre codeur de conception analogue et de même dimension, construit également sur une piste unique, transmet une information sur le nombre de tours du premier codeur. Selon le nombre de rangs du second codeur la gamme de mesure peut être étendue à 2 ou 4 m tout en conservant une précision de 1 cm.

Il est important de remarquer que la transmission et le décodage du code du premier capteur (exécutant plusieurs cycles) et du second capteur qui compte le nombre de cycles s'effectuent indépendamment l'un de l'autre. Un dispositif logique simple, situé au centre de réception, permet d'éliminer l'effet d'"indétermination des frontières" lors du passage du niveau "99" au niveau "00" et réciproquement. La valeur numérique de la position du premier capteur donne les unités et les dizaines de la grandeur mesurée, tandis que l'information transmise par le second capteur donne les centaines et les milliers.

Grâce à cette particularité, des capteurs à gamme de mesure différente peuvent fonctionner dans un système commun, tout en utilisant un centre de réception commun. L'erreur maximale absolue de mesure pour tous les capteurs lors de la mesure du niveau ne dépasse pas le centimètre, ce qui correspond aux exigences des entreprises hydrotechniques.

Pour la mesure du niveau, comme il a été dit plus haut, en guise d'élément sensible ou utilise un flotteur qui est équilibré par un contrepoids. Pour la mesure du débit par différence de niveaux, on utilise deux longs cylindres dont le poids spécifique dépasse celui de l'eau ; ces cylindres pendent sur un filin d'acier passé sur la poulie du capteur, et chacun d'eux est immergé dans l'un des biefs dont il faut mesurer la différence.

La force de poussée agissant sur les deux cylindres les oblige à occuper une position d'équilibre qui correspond à la différence de niveaux.

Pour la mesure de la position des vannes, l'axe du codeur est relié soit directement à la vanne, soit à sa vis de levage.

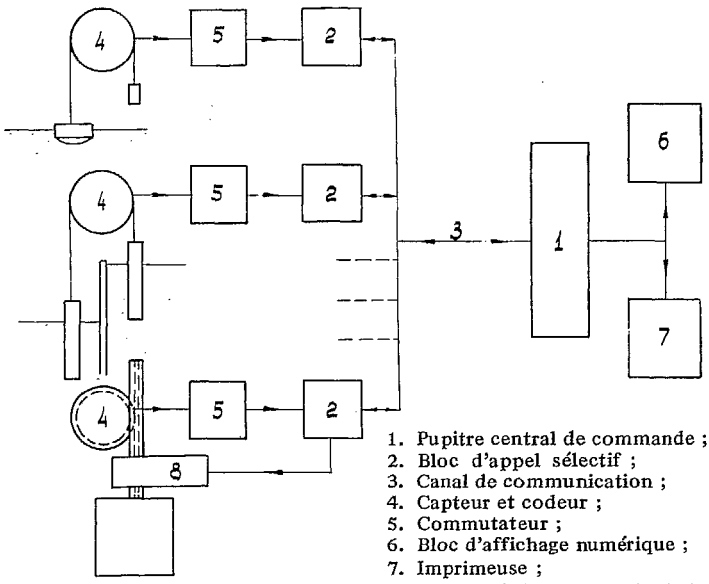
Selon le caractère de la grandeur mesurée, l'unité de mesure peut être le centimètre (pour la mesure du niveau d'eau ou de la position d'une vanne), le millimètre (pour la mesure de la différence de niveaux) ou le pourcentage (pour la mesure, dans certains cas, de la position des vannes).

Sur le pupitre de commande, l'affichage des chiffres reste inchangé, seule l'unité de mesure est automatiquement indiquée. Ces unités sont introduites d'avance dans le pupitre de commande, lors de l'installation du système, en plaçant les circuits correspondants dans le dispositif de l'adresse.

Le dispositif est conçu pour fonctionner sur fils téléphoniques. Son rayon d'action est de 80 km. Dans ces conditions l'alimentation s'effectue à distance à partir du centre.

Les capteurs peuvent en principe fonctionner avec d'autres canaux de transmission (par exemple, par radio).

Lors du travail sur réseau téléphonique par l'intermédiaire du système de télécommande "Temir", à une paire de fils téléphoniques peuvent être branchés jusqu'à 48 postes de contrôle et à un pupitre de commande peuvent être connectées jusqu'à



1. Pupitre central de commande ;
2. Bloc d'appel sélectif ;
3. Canal de communication ;
4. Capteur et codeur ;
5. Commutateur ;
6. Bloc d'affichage numérique ;
7. Imprimeuse ;
8. Dispositif de commande de la vanne.

Figure 1 – *Schéma-bloc du dispositif*

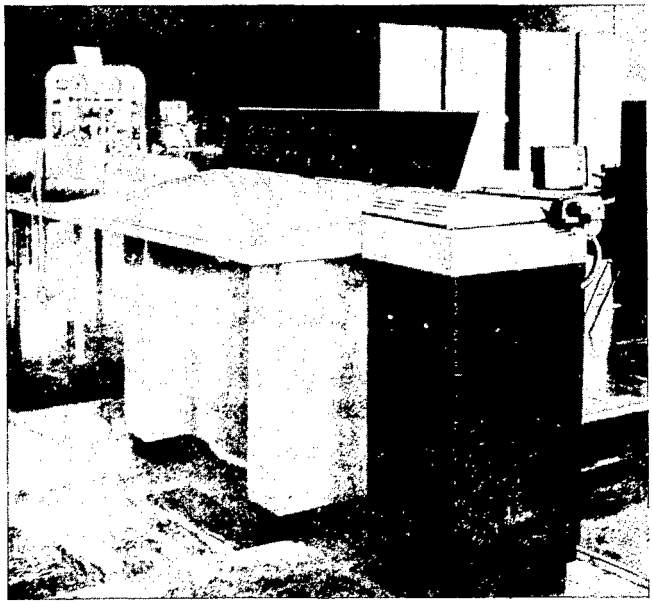


Figure 2 – *Vue générale du pupitre de commande et du bloc d'enregistrement numérique*

10 paires de fils. Ainsi le nombre maximal de postes contrôlables à partir d'un centre est de 480, ce qui dépasse de beaucoup les exigences des systèmes d'irrigation, même les plus grands.

La figure 1 représente le schéma-bloc du dispositif.

Le pupitre central de commande "1" est relié aux blocs d'appel sélectif "2" des postes de contrôle, par l'intermédiaire du canal de communication "3".

Sur une commande transmise du centre de commande, le capteur voulu avec son codeur "4" sont branchés par l'intermédiaire du commutateur "5" qui transmet dans le réseau l'une après l'autre les impulsions du code.

Le décodage de ces signaux ainsi que l'affichage numérique sur le bloc "6" s'effectuent automatiquement au centre de commande.

En outre, les résultats de mesure sont enregistrés à l'aide d'une imprimeuse "7".

Lorsqu'une commande de manœuvre est émise par le centre, le dispositif de commande de la vanne "8" est branché par l'intermédiaire du même bloc d'appel sélectif "2".

Un système de codage permet d'effectuer à partir du centre l'appel sélectif de n'importe quel poste pris à part ou de tous les postes périodiquement l'un après l'autre. Dans le premier cas, l'affichage de la grandeur mesurée s'effectue sur un tableau numérique, dans le second cas, en plus de l'affichage, un enregistrement automatique des données de mesure s'effectue à l'aide d'une imprimeuse électrique. Dans ce cas, pour augmenter la fiabilité de transmission de l'information, chaque paramètre est transmis et enregistré deux fois.

Le temps d'appel du poste est de 0,2 seconde. Lors de l'appel périodique avec enregistrement automatique, le temps d'une période de contrôle de 48 stations, y compris l'enregistrement des résultats de mesure, est d'environ de 2 minutes.

Lors du fonctionnement des 10 paires de fils téléphoniques, le temps de contrôle de 480 postes est d'environ 20 minutes.

Le dispositif "Temir" a été exposé à l'exposition internationale Automatizacia 69 à Moscou. La figure 2 représente la vue générale du pupitre de commande et du bloc d'enregistrement numérique.

En plus du télécontrôle numérique, ce dispositif permet de transmettre des signaux d'"alarme". Pour cela, les postes de contrôle doivent être munis de signalisateurs de cote d'alarme qui établissent un contact lorsque le niveau atteint une cote préalablement fixée. De tels contacts sont montés sur le capteur décrit plus haut et peuvent être réglés sur place à la cote voulue.

Le signal d'alarme est transmis au centre de commande dès son apparition. La recherche et l'affichage du poste d'où provient ce signal s'effectuent automatiquement.

Le système peut de même transmettre des signaux de télécommande des vannes ou d'autres dispositifs de manœuvre. Lorsque les signaux de télécontrôle et de télécommande sont absents sur le réseau, celui-ci peut être utilisé pour des liaisons téléphoniques. Pour cela des appareils habituels à batterie locale sont complétés de blocs spéciaux grâce auxquels s'effectue leur appel sélectif à partir du centre.

Le pupitre de commande du système permet l'introduction de l'information numérique de mesure dans une calculatrice électronique.

Recording and teletransmission of measured data in hydrology and relevant WMO activities

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Abstract. The Working Group on Instruments and Methods of Observation of the WMO Commission for Hydrometeorology (CHy) was requested to prepare a technical note on automatic equipment for observing and transmitting hydrometeorological elements. The scope of this work, including the description and evaluation of automatic sensors, teletransmitting and automatic processing equipment, is described in this paper. While particular emphasis is placed on elements connected with hydrometry, other hydrological elements are also considered. Concise general information on WMO activities in the whole field of automatic collection, teletransmission and processing of data is also given.

ENREGISTREMENT ET TELETRANSMISSION DES MESURES DES DONNEES HYDROLOGIQUES ET ACTIVITES DE L'OMM DANS CE DOMAINE

Résumé. Le groupe de travail sur les instruments et les méthodes d'observation de la Commission d'hydrométéorologie de l'OMM a été prié de préparer une note technique sur les équipements automatiques utilisés pour l'observation des éléments hydrométéorologiques et la transmission des données ainsi obtenues. La communication définit le domaine d'étude en question, qui englobe notamment une description et un examen critique des instruments de mesure automatiques, ainsi que de l'équipement de télétransmission et de traitement automatique des données. Si les éléments se rapportant à l'hydrométrie retiennent tout particulièrement l'attention, d'autres éléments hydrologiques sont également examinés. La communication contient également de brefs renseignements d'ordre général sur les activités déployées par l'OMM dans l'ensemble du domaine du rassemblement, de la transmission et du traitement automatique des données.

REGISTRO Y TELETRANSMISIÓN DE DATOS HIDROLÓGICOS Y ACTIVIDADES DE LA OMM

Resumen. El grupo de trabajo de instrumentos y métodos de observación de la Comisión de Hidrometeorología de la OMM ha sido encargado de preparar una nota técnica sobre los equipos automáticos utilizados para la observación de los elementos hidrometeorológicos y la transmisión de los datos así obtenidos. La comunicación define el campo de estudio en cuestión, que engloba especialmente una descripción y un examen crítico de los instrumentos de medida automáticos así como del equipo de teletransmisión y de tratamiento automático de los datos. Aunque los elementos que se refieren a hidrometría se estudian con mayor atención, se examinan también otros elementos hidrológicos. La comunicación contiene también breves informaciones de orden general sobre las actividades desplegadas por la OMM en el conjunto toma de datos, transmisión y tratamiento automático.

ЗАПИСЬ И ТЕЛЕПЕРЕДАЧА ИЗМЕРЕННЫХ ДАННЫХ В ГИДРОЛОГИИ И СООТВЕТСТВУЮЩАЯ ДЕЯТЕЛНОСТЬ ВМО

Резюме. Рабочая группа по приборам и методам наблюдений комисии ВМО по гидрометеорологии попросила подготовить техническую записку по автоматическому оборудованию для наблюдений и передачи гидрометеорологи-

ческих элемеитов. В этой статье описан объем этой заботы, включая описание и расчет автоматических датчиков, оборудование для телепередачи и автоматической обработки. Хотя особый упор делается на элементы, связанные с гидрометрией, в статье также рассматриваются и другие гидрологические элементы. Кроме того, дана краткая информация о деятельности ВМО в области автоматического сбора, телепередачи и обработки данных.

1. The WMO Commission for Hydrometeorology (CHy) was established in 1959 by the Third Congress of WMO. From the start of its activities the Commission gave special attention to operational hydrology. Since hydrometry is a large subject of direct concern to operational hydrology, WMO was pleased to co-sponsor this symposium and to sponsor several papers on specific subjects. This paper gives a general review of WMO's work in the field of instruments, with particular emphasis on hydrometry.

2. In 1967 the Fifth Congress reviewed the tasks of CHy and included among them the following:

Developing, improving and promoting the international standardization of methods, procedures, techniques and terminology for (a) studies of the water balance, the global hydrologic cycle and hydrological forecasting, and (b) hydrometeorological aspects of design of systems for water management and control;

Considering questions relating to the reliability and homogeneity of hydrometeorological observations;

Standardizing methods of computation of hydrometeorological data for research and publication (means, ranges, frequencies, etc.);

Considering questions relating to instruments and methods of observation.

WMO produces numerous publications by which the results of its activities are made available to its Members and to the large community of specialists. These publications can be grouped as follows:

Technical Regulations, which formulate standard practices and procedures which have to be followed by Members, and recommend practices and procedures which should be followed by Members;

Guides and manuals, describing in a more detailed manner practices and procedures which Members are invited to follow;

Technical Notes for operational and educational purposes;

Other technical publications and reports, partly edited in collaboration with other organizations.

CHy has already prepared a great number of such publications, aiming to meet the requirements of its terms of reference. The following publications are particularly relevant to the subject of this symposium:

Guide to Hydrometeorological Practices
Measurement of peak discharges by indirect methods
Instruments and measurements in hydrometeorology
Estimation of maximum floods
Hydrological network design-needs, problems and approaches.

3. At its third session (1968) CHy established nine working groups, among them groups on: instruments and methods of observation; machine processing of hydrometeorological data; water level and stream discharge measurement; hydrological aspects of the World Weather Watch (WWW).

Several rapporteurs were asked to report to the next session on particular questions. One rapporteur was appointed to collect recommendations from Members regarding instruments in the field of hydrometeorology which they have found reliable in specified environments, and to consolidate these recommendations in a report for the use of Members and others in establishing networks.

The Working Group on Hydrological Aspects of the WWW is particularly concerned with the application of meteorological satellites, high-speed computers and modern telecommunications to ways in which they can benefit operational hydrology and hydrological forecasting in particular. The Working Group on Machine Processing of Hydrometeorological Data has finalized a Technical Note on this subject; it is expected to be published during 1970.

4. In connexion with the work of the Rapporteur on Instruments, WMO has sent its Members a questionnaire on hydrometeorological instruments of proved reliability. This survey has been conducted to collect recommendations on instruments for hydrological purposes for which there is little guidance available. The recommendations will be consolidated in a report. The replies to the questionnaire shall deal only with those instruments which have been tested and proved reliable under actual operational conditions and concerning the following hydrometeorological parameters:

Rainfall intensity;
Water equivalent and density of snow pack (not snow fall);
Meteorological elements of floating stations for energy balance or mass transfer estimates of evaporation;
Soil moisture, including moisture in frozen soils but excluding lysimeters;
River and lake stage;
Discharge measurements;
Water temperature in streams and lakes;
Suspended sediments.

5. The terms of reference of the Working Group on Instruments and Methods of Observation specify the following tasks:

To complete the drafting of a Technical Note on automatic equipment for observing and transmitting hydrometeorological elements, and to prepare a summary of this Technical Note suitable for inclusion in the Guide to Hydrometeorological Practices;
To take any necessary action to revise and complete the material for the Guide on snow measurements, lysimeters, transpiration and evaporation from the soil, prepared by other Commissions of WMO;
To prepare a comprehensive report and critical appraisal of work already accomplished within WMO in the field of standardization of instruments and methods in hydrometeorology;
To examine the need and feasibility of comparison of further instruments and methods in hydrometeorology, in addition to work already under way in WMO;
To examine if some of the above instruments and methods are already suitable for international standardization;
To recommend a work plan for further activities within WMO in the field of standardization of hydrological instruments and methods of observation.

In preparing the above Technical Note on automatic equipment for observing and transmitting hydrometeorological elements, a large number of national hydrological organizations and instrument manufacturers were requested to provide technical

documentation on achievements in this field. More than a hundred papers were collected as a result of this inquiry. These have been analysed and a summary of the existing methods for automatic measurement, conversion and transmission of hydrological data is being prepared. The material collected so far gives information on the automatic sensing of a range of important hydrological elements (amount and intensity of precipitation, water level, flow rate, water equivalent of snow, water surface temperature, suspended sediment transport).

Many meteorological elements commonly observed at meteorological stations, and which are observed and recorded automatically, will be included in the Technical Note. It is already evident that some elements, both hydrological and meteorological (such as wave characteristics) have been neglected from the point of view of automation.

Examples of sensing and converting principles in use for some of the main elements are given below. It should be noted that the summary only covers a small part of the material collected for the Technical Note.

Water level. The level variations in streams measured by means of floats and pressure devices can be converted for automatic recording in many ways, of which the conventional chart diagram is one. For transmission of electrical pulses the float movement or the pressure may work on the movable case of a transformer. Another method utilizes a potentiometer, which in turn may be coupled to an audio-frequency generator. Still another method, which can be applied to several other elements than water level, uses a weighing balance beam which closes electrical contacts at the tip when in unbalance. A motor then drives a poise to re-establish balance.

Flow rate. Only one method developed recently will be mentioned here, namely the transmission of ultrasonic pulses across the stream. The travel time is a function of the streamflow velocity, which can be calculated when the sound velocity in still water and the path length are known. In order to obtain the mean velocity for a particular cross-section, the velocity in several paths should be observed. At present the method has only limited application because of the lack of effectiveness in irregular channels.

Water surface temperature. The use of airborne radiation thermometers is the most promising technique, based on the fact that the flux of radiating energy is greatest in the infra-red part of the spectrum, where also the attenuation of radiation by water vapour and carbon dioxide is small (between 8 and 14 microns). The infra-red thermometer senses the radiant energy from the water surface and from a black reference body. The difference between the two radiations is converted to an electrical signal which varies with the temperature of the water surface.

6. Using the material already collected, a synthesis is now being prepared. The preparation of the Technical Note will take into account the following general considerations:

The Note will be consulted by services and experts having to install stations and buy instrumental equipment. They will have to ask for tenders from manufacturers and to select from amongst suggested solutions. The Technical Note has to give the necessary guidance for this purpose. The reader must find in the Technical Note descriptions of: general operation of the different types of instruments, conditions under which a certain type can be used, limits of applicability of each type, economic factors, needs for trained personnel, and needs for power supply.

The number of hydrological elements to be considered in the Technical Note will have to be limited to the most important ones mentioned above in the questionnaire

drawn up by the CHy Rapporteur on Instruments. In addition, some meteorological parameters, important for hydrology, will be considered.

The elements preliminarily considered for treatment in the Technical Note can be grouped as follows :

First priority hydrological elements: intensity of precipitation, water equivalent of snow, potential evaporation from free water surface, river and lake stage, stream-flow velocity, discharge, water surface temperature, and suspended sediment.

Second priority elements: amount of precipitation, snow depth, current direction, water temperature below the surface, ice thickness and density, wave elements (height, length, etc.), and soil moisture.

Meteorological elements of hydrological importance: air humidity and dew point, formation of hoar-frost, atmospheric pressure and pressure gradients, wind velocity and direction, air temperature.

7. For the different elements treated in the Technical Note, the systems for sensing and transmission of data will be described. The descriptions will include technical details about :

Measurement: sensor, unit, range, accuracy, timing, display, power supply.

Conversion: conversion type (ex. analogue, digital), signal, manual input, coding.

Transmitting: pulse and frequency characteristics, transmitting media (telephone, telegraph, television).

Transfer line: e.g. telephone cable, phantom circuit, signalling cable, high-voltage line, radio transmission, satellite.

Received signal: DC-AC, modulation, radio wave lengths, amplification, rectifying, conversion (ex. digital-digital), decoding, storage.

Receiving device: e.g. analogue or numerical indicators, acoustics, diagrams, printers, perforators, magnetic recorders.

A draft of the Technical Note will be examined by the members of the Working Group during 1970. In 1971 a final draft will be prepared, the publication of which is foreseen in 1972. It is hoped that this Technical Note will prove useful to hydrological services in countries all over the world and, in particular, for those in developing countries.

XII Evaluation of measuring values

Évaluation des mesures obtenues

Appareillage pour l'enregistrement et le traitement de l'information dans le domaine des stations de jaugeage et des stations-usines

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Résumé. La Division technique générale du Service de la production hydraulique d'Electricité de France a mis en place et exploite depuis quinze ans un réseau hydrométrique de stations de jaugeage et de stations-usines, pour répondre aux besoins des services d'exploitation.

Le calcul des débits des rivières est effectué par transformation des données de l'enregistrement graphique des hauteurs d'eau soit par report simple sur un barème, soit à l'aide d'un appareil analyseur de limnigramme semi-automatique, imprimant la valeur des débits; cet appareil a été construit en 1961. Les valeurs de débit sont ensuite traitées sur un calculateur qui effectue tous les contrôles, corrections et calculs annexes et restitue des résultats prêts à la publication.

A partir de 1970, une autre méthode a été utilisée : l'emploi d'un analyseur de limnigramme perforateur (ADLP 68) spécialement construit, permet de restituer les hauteurs enregistrées sous forme de bandes perforées ; celles-ci sont traitées directement et totalement par le calculateur.

Une méthode de dépouillement entièrement automatique est en cours de préparation : il est prévu d'adjoindre à chaque limnigraphe un codeur enregistreur incrémental sur bande magnétique en cassettes et de transférer les données ainsi enregistrées sur bande magnétique assimilable par le calculateur.

EQUIPMENT FOR RECORDING AND TREATING INFORMATION IN THE FIELD OF GAUGING STATIONS AND POWER STATIONS

Abstract. In order to meet the needs of the operating departments, the General Technical Division of Electricité de France's Hydraulic Generating Department set up and has been running for 15 years a hydrometric network of measuring stations and power stations.

Rates of flow in rivers are calculated from graphical recordings of water levels, either by simply reading off a table or by means of a semi-automatic level recording analyser built in 1961 which prints out discharge values. These are then processed on a computer which makes any necessary checks, corrections and secondary calculations, and prints out the results ready for publication.

As from this year (1970), another method will be used: a specially built perforating level recording analyser (ADLP 68) will deliver the level recordings in the form of punched tape which will be fed directly to the computer for complete processing.

A fully automatic method of treatment is being prepared. It is proposed to fit each level recorder with an incremental coder recording on magnetic tape in cassette form, and to transfer the data obtained on a magnetic tape capable of being fed to the computer.

EQUIPO PARA EL REGISTRO Y TRATAMIENTO DE LA INFORMACIÓN EN EL CAMPO DE LAS ESTACIONES DE MEDICIÓN Y CENTRALES ELÉCTRICAS

Resumen. A fin de satisfacer las necesidades de los departamentos de explotación, la División Técnica General del Departamento de Producción Eléctrica de Electricité de France estableció hace quince años, y desde entonces se encuentra en marcha, una red hidrométrica de estaciones de medición y medición en centrales.

El valor del caudal en los ríos se calcula a partir de registros gráficos de niveles de agua, bien por simple lectura o bien por medio de un analizador de registro de nivel semiautomático, construido en 1961, que imprime los valores del caudal. Estos se procesan a continuación en un ordenador que realiza análisis, correcciones y cálculos secundarios que puedan ser necesarios y que imprime los resultados listos para su publicación.

A partir de 1970, se ha utilizado otro método: un analizador especialmente construido de tipo registrador de nivel perforador (ADLP 68) proporciona los registros de nivel en forma de cinta perforada que alimentan directamente al ordenador para su proceso completo.

Se está preparando un método de tratamiento totalmente automático. Está destinado a equipar cada registrador de nivel con un código adicional que registra sobre una cinta magnética en forma de cassette, y transmite los datos obtenidos a una cinta perforada para su utilización en el ordenador.

ПРИБОРЫ ДЛЯ РЕГИСТРАЦИИ И ОБРАБОТКИ ДАННЫХ НА ИЗМЕРИТЕЛЬНЫХ СТАНЦИЯХ И ГЭС

Резюме. Для удовлетворения запросов оперативных управлений Общий технический отдел Департамента Гидравлической энергии при Electricité de France организовал 15 лет тому назад гидрометрическую сеть измерительных станций и ГЭС, которая с тех пор функционирует.

Величины стока в реках вычисляются с помощью графической записи уровней воды, или путем простой записи в таблицу или посредством полуавтоматического регистратора уровней, построенного в 1961 г., который печатает значения расхода. Эти данные затем обрабатываются на компьютере, который делает все проверки, исправления и дополнительные вычисления, которые могут потребоваться, и печатает результаты, годные для публикации.

Накиная с этого года (1970), будет использоваться другой метод: специально построенний перфорационный регистратов уровней (ADLP 68) даст показания уровней в форме перфоленты, которая будет непосредственно вводиться в компьютер для полной обработки.

Разрабатывается полностью автоматизированный способ обработки. Предполагается снабдить каждий регистратор уровней кодирующим устройством, делающим записи на магнитной ленте в форме кассеты и передающии полученные данные на магнитную ленту, которую можно вводить в компютер.

CARACTERISTIQUES DU RESEAU HYDROMETRIQUE EXPLOITE PAR LA DIVISION TECHNIQUE GENERALE

Le réseau hydrométrique exploité

Le réseau hydrométrique exploité depuis plus de quinze ans par la Division technique générale du Service de la production d'Electricité de France, a été mis en place pour répondre aux besoins particuliers de l'exploitation.

Il présente, de ce fait, certaines particularités : conception, exploitation, nature des informations recueillies et élaborées, fréquence de leur diffusion.

Nature de l'information et moyens d'acquisition

On distingue l'information élémentaire, directement recueillie aux divers points de mesure (par enregistrement, relevés d'indicateur de position de vanne, etc.) et l'information élaborée (par calcul ou interprétation) à partir de celle-ci dans les centres de traitement.

Information élémentaire

Actuellement, trois moyens sont essentiellement utilisés, suivant la nature de l'information.

Stations de jaugeage. Ecoulements à surface libre en rivière, galerie et canaux, où l'information recueillie est la variation de la hauteur d'eau dans le temps.

Les courbes de tarage hauteur d'eau – débits de ces stations sont établies, puis contrôlées et éventuellement modifiées par des jaugeages instantanés faits par la méthode de dilution ou au moulinet pour différentes valeurs du débit.

Ces stations sont munies de limnigraphes qui enregistrent la hauteur d'eau en utilisant le plus souvent des flotteurs se déplaçant dans des puits en communication avec la rivière, mais on trouve aussi des limnigraphes à bulles utilisant une mesure de pression, qui présentent l'avantage de la simplicité du génie civil d'implantation et une moins grande sensibilité à l'envasement.

Par souci de standardisation et en prévision d'un traitement mécanographique ultérieur, le réseau a été, dès l'origine, équipé d'un seul type d'enregistreur classique dont l'autonomie peut atteindre un an (par mouvement d'horlogerie remonté par micromoteur à piles sèches, logées en caisson étanche pouvant être immergé dans la rivière pour protéger les piles des effets des basses températures) et dont le mode d'enregistrement utilisé est la mine de graphite sur papier, ou mine d'argent sur papier baryté.

De plus, environ 45 stations d'intérêt particulier sont équipées de limniphones, répondeurs automatiques permettant d'obtenir par simple appel téléphonique depuis n'importe quel point du réseau des Postes et télécommunications, et directement en langage clair, la cote de la rivière au moment de l'appel et les cotes aux dix heures précédentes.

Remarques. La connaissance des débits déversés ou restitués dans les aménagements hydro-électriques peut être obtenue par station de jaugeage dans certains cas, ou, suivant la nature des ouvrages (vannes levantes ou déversantes, à jet creux ou à diaphragmes, etc.) faire l'objet de mesures telles que l'enregistrement de la cote du plan d'eau et de la position d'un organe de commande de l'ouverture.

Débits turbinés ; stations-usines. Dans les centrales hydro-électriques, le calcul des débits turbinés est le plus souvent effectué à partir de la production d'énergie et des caractéristiques de l'usine (hauteur de chute, pertes de charges dans les ouvrages d'amenée, rendements des turbines et des alternateurs). Les renseignements permettant directement un calcul précis du débit moyen journalier turbiné sont récapitulés par les exploitants sur des imprimés spéciaux communiqués hebdomadairement à la Division technique générale. On trouve cependant dans certains cas particuliers des appareils intégrateurs de débit utilisant soit des lois débits – différence de pression, soit des lois débits – ouverture de la turbine.

Débit équivalent de variation de réserve ; barrages-réservoirs. Le "débit équivalent de variation de réserve" (grandeur algébrique positive lorsque la réserve croît, négative lorsqu'elle décroît) est pratique pour l'établissement des bilans d'aménagements et pour la reconstitution précise des débits naturels. Il est déterminé à partir de relevés de la cote du plan d'eau et du barème de capacité du réservoir.

Ces barèmes sont établis de centimètre en centimètre, par intégration sur calculateur de la courbe des surfaces de la réserve en fonction de la cote, courbe obtenue d'après des relevés par stéréophotogrammétrie aérienne de précision.

Information élaborée

Apports aux aménagements. Dans le cas d'un aménagement hydro-électrique comportant une usine et un réservoir, les débits turbinés, déversés et de variation de réserve constituent une information fragmentaire intéressante, mais souvent insuffisante. On définit ainsi le débit d'apports : débits d'apports = débit turbiné + débit déversé + débit de variation de réserve.

Débits influencés et débits naturels reconstitués. Les aménagements hydro-électriques modifient les débits qui s'écouleraient sans leur présence dans le lit des rivières.

Certaines études et contrôles nécessitent la connaissance de séries de débits homogènes, donc en pratique la reconstitution de débits naturels.

Lorsque le débit déduit des mesures effectuées en un point d'une rivière est influencé par les aménagements situés à l'amont, on définit le débit naturel reconstitué en ce point comme la somme du débit influencé et d'un débit d'influence amont.

Le débit d'influence est une grandeur algébrique, définie comme la somme à l'intérieur du bassin versant relatif au point de mesure, des débits de variations de toutes les réserves (positifs ou négatifs), des débits d'adductions en provenance d'autres bassins (négatifs) et des débits de dérivations vers d'autres bassins (positifs).

Ces calculs longs et fastidieux sont effectués, en général, sur les valeurs moyennes mensuelles, seuls quelques cas particuliers pouvant être traités à échelle journalière.

Diffusion de l'information

Les besoins courants de l'exploitation imposent, pour une utilisation immédiate, une diffusion rapide, en général mensuelle, des informations fournies par le réseau. Leur précision ne doit pas être affectée par la rapidité de leur élaboration, souvent assez complexe.

Certaines études, enfin, nécessitent un accès aisé aux archives hydrométriques et le traitement rapide d'un volume important de données.

Ces impératifs ont conduit la Division technique générale à améliorer de façon continue ses méthodes de traitement de l'information en recherchant simultanément sécurité, précision et rapidité.

Moyens et méthodes ont donc évolué, mais la recherche d'une grande souplesse dans le traitement, la décentralisation de celui-ci (trois centres régionaux) et la nécessité de toujours disposer de moyens de secours font parfois coexister, à chaque étape, des systèmes de conception et de performances différentes.

TRAITEMENT DE L'INFORMATION

Un calculateur électronique a été utilisé à partir de 1965. On peut distinguer deux étapes principales dans l'évolution du traitement.

Avant utilisation du calculateur

L'effort principal a porté, depuis la création du réseau, sur la normalisation et le perfectionnement de moyens et de méthodes traditionnels (dossiers de calculs des débits turbinés, barèmes normalisés de capacité des réservoirs, processus d'analyse des enregistrements, contrôles types des résultats de débit, etc.).

La transformation des hauteurs d'eau enregistrées en débits est faite encore dans de nombreux cas par un technicien qui relève sur le diagramme les valeurs de la hauteur d'eau toutes les deux heures ou toutes les heures, leur fait correspondre, à l'aide du barème hauteur-débit de la station, 12 ou 24 valeurs de débit dont il fait la moyenne pour obtenir le débit moyen journalier.

Ce procédé a été amélioré depuis 1961 par l'utilisation, à Grenoble, d'un analyseur semi-automatique de limnigrammes, l'ADL 61, conçu et réalisé par la Division technique générale. Cet appareil assure la transformation hauteur-débit par analogie électrique à partir de la courbe de tarage de la station qui est introduite par un jeu de

16 potentiomètres. Les limnigrammes sont placés sur un tambour dont la rotation s'effectue à un pas prédéterminé correspondant à l'échelle des temps. Après avoir à chaque pas assuré manuellement la coïncidence d'un suiveur se déplaçant sur l'échelle des hauteurs avec le tracé de l'enregistrement, l'opérateur déclenche un processus qui assure l'avance d'un pas de temps, le calcul et l'impression du débit correspondant, puis, à la fin de chaque journée, l'enregistrement du débit moyen journalier.

Ce dispositif a permis d'améliorer la précision et la rapidité de l'exploitation, longue et fastidieuse, des enregistrements de hauteur d'eau.

Utilisation du calculateur électronique

Première phase

La Division technique générale a pu disposer, dès 1965, d'un ordinateur de type CAE 510 (mémoire centrale de 16 000 mots, 2 dérouleurs de bandes magnétiques, entrée-sortie sur ruban perforé).

Un ensemble de programmes qui traite les données de débits obtenues par les procédés décrits plus haut put, dès lors, être mis au point pour améliorer et accélérer le traitement des informations du réseau[1].

Résultats primaires. Un ensemble de programmes traite les résultats primaires (débits de stations de jaugeage et de stations-usines, volume des réserves) en assurant l'homogénéité des données, le calcul des moyennes, une certaine critique fondée sur des règles simples, et les calculs annexes (débit spécifique, lame d'eau, coefficient d'hydraulicité, etc.). Sa conception permet le traitement mensuel des informations, qui sont ensuite archivées sur bandes magnétiques.

Résultats secondaires. Un programme particulier d'addition réalise, à échelle annuelle, le calcul des résultats secondaires (apports aux aménagements, débits influencés et débits d'influences, débits naturels reconstitués, bilans de contrôle des bassins intermédiaires entre deux points de mesures successifs sur une même rivière, etc.) à partir des résultats primaires déjà traités et des formules d'addition correspondantes ; ces résultats secondaires sont eux-mêmes archivés avec les résultats annexes nécessaires.

Programmes particuliers. Programme de calcul des tableaux pluriannuels de moyennes mensuelles et annuelles, permettant la mise à jour rapide des moyennes de référence.

Programme de correction automatique. La complexité des calculs de reconstitution de débits naturels pour les bassins versants équipés de nombreux aménagements hydro-électriques a conduit à établir un programme assurant le report automatique des corrections, entraînées sur tous les tableaux concernés par la modification de données dans un tableau de débits amont. Ce programme fait intervenir l'ensemble des formules d'addition (journalières et mensuelles) utilisées pour les résultats secondaires. Il permet de réaliser sans omission les multiples corrections entraînées, par exemple, par le changement de courbe de tarage d'une station mesurant le débit dans une galerie de dérivation d'un bassin dans un autre, etc.

Publication des résultats : elle est réalisée, par programme, sous forme de tableaux complets, directement sur des calques préimprimés, évitant les erreurs de recopie et assurant une présentation de bonne qualité (cadrage et arrondis automatiques).

1. Voir la thèse d'ingénieur docteur de M. Fleure, *Traitement des données de débits sur calculateur électronique*, Faculté des sciences de Grenoble, 1968.

Cet ensemble de programmes a conduit à une amélioration certaine de la qualité des résultats publiés et archivés ; par ailleurs, il a permis la création d'un fichier sur bandes magnétiques utilisable directement pour les études annexes par ordinateur.

Seconde phase

But. Dès le début de l'utilisation de l'ordinateur, il est apparu que le système d'analyse des enregistrements de hauteur d'eau par l'appareil ADL 61 se prêtait mal à un traitement rationnel. Il oblige, en effet, à reprendre en mécanographie les informations fournies par l'analyseur pour le traitement par l'ordinateur, ce qui est une source d'erreur et alourdit le contrôle des données ; par ailleurs, la constatation d'un détarage de station lorsqu'elle intervient après analyse nécessite un nouveau passage complet des enregistrements à l'analyseur.

Pour ces raisons, la Division technique générale a mis au point un système plus complet de traitement appliqué à partir de 1970. Il a été rendu possible par la mise à notre disposition d'un ordinateur plus puissant (ICL 1902 à mémoire magnétique sur disques, disposant d'une imprimante rapide et d'entrées-sorties sur cartes, ruban perforé et ruban magnétique) et la mise au point d'un analyseur de limnigrammes perforateur.

L'analyseur de limnigrammes - perforateur ADLP 69. Certains services hydrologiques utilisent des appareils très élaborés, de possibilités étendues mais assez coûteux, qui effectuent, de façon automatique ou semi-automatique, la transformation des enregistrements analogiques en données numériques, codés soit sur ruban ou cartes perforées, soit sur bandes magnétiques. Ces appareils existent dans le commerce.

Il est apparu préférable pour réaliser l'analyse des limnigrammes de concevoir un dispositif plus simple, de coût moins élevé, afin d'en doter chacun des centres régionaux de calcul ; il comprend une table qui reçoit le limnigramme et un ensemble perforateur mécanique OTT. Le système de pointage de la courbe qui équipe la table est inspiré directement de celui de l'ADL 61 : avance semi-automatique d'une règle à un pas prédéterminé sur l'axe des temps et commande manuelle d'une seconde règle sur l'axe des hauteurs par l'opérateur qui, après avoir assuré la coïncidence de la courbe et de l'intersection de l'axe des deux règles, commande la perforation. Celle-ci est réalisée en code CCITT n° 2 sur ruban perforé à 5 canaux ; l'identification des bandes perforées est assurée par la perforation au début de l'analyse et au moyen de l'appareil lui-même d'une série de valeurs codées (affichées comme des hauteurs).

Ces bandes peuvent ensuite, soit être utilisées directement par un programme de traitement sur ordinateur, si celui-ci est muni d'un lecteur de ruban 5 canaux, soit recopiées sur bandes magnétiques au moyen d'un appareil existant dans le commerce qui permet, en outre, l'addition d'informations complémentaires par clavier.

Transformation hauteur-débit. Celle-ci est assurée automatiquement par un programme recevant en données les bandes issues de l'ADLP 69 et utilisant un fichier de courbes de tarage. Ce programme effectue le contrôle des hauteurs perforées et de l'identification, restitue les bandes incorrectes, traite les autres en calculant les débits moyens journaliers, les débits moyens mensuels, le débit correspondant à la donnée maximale de hauteur d'eau entrée pour chaque période. Il accepte des bandes de données de longueur quelconque, mais à "pas de temps" constant. Les résultats sont écrits sur bande magnétique, mais peuvent également être édités sur imprimante pour une utilisation immédiate.

Fichier de courbes de tarage. Il est enregistré sur disques magnétiques, ce qui permet un accès très rapide et direct aux courbes concernant chaque station, et est décomposé en sous-fichiers annuels ; chaque courbe est archivée sous la forme d'une série de couples hauteurs-débits, identifiant des points sélectionnés sur la courbe tracée par interpolation graphique manuelle à partir des jaugeages. Les procédures d'interpolation numériques sont incorporées dans les programmes utilisant le fichier, ce qui permet une grande souplesse dans leur choix.

Fichier hydrométrique et programmes de traitement. Le système de traitement a été conçu pour être mis en œuvre à des fréquences quelconques, accepter des données de présentations diverses le plus proches possible de leur forme initiale d'élaboration, traiter des périodes de longueur variable, afin d'assurer l'archivage des données de débits au fur et à mesure de leur élaboration et la possibilité d'éditer à tout moment tout ou partie du fichier dans l'état où il se trouve.

Il comporte un programme général de traitement des données primaires (stations de jaugeage, usines, réserves) qui effectue la critique, l'archivage et la publication provisoire de périodes de données de un jour à un an, qui peuvent être présentées sous différentes formes : bande magnétique, cartes ou ruban perforé ; débits résultant du traitement des hauteurs d'eau par le programme de calcul ou des calculs traditionnels (usines, limnigrammes particuliers, réserves, etc.) ; volumes ou variations journaliers des réserves.

Un programme de calcul des résultats élaborés, à fréquence mensuelle, le complète (addition de tableaux, débits d'influence, bilans, etc.).

Enfin, les différents programmes annexes déjà utilisés avec l'ordinateur CAE 510 ont été conservés : publication de tableaux complets, correction automatique, courbes de débits classés, etc.

TRAITEMENT AUTOMATIQUE EN PREPARATION

Utilisation de codeurs-enregistreurs magnétiques dans les stations de jaugeage et les stations-usines.

Les dernières années ont vu la généralisation dans certains services hydrologiques[1] de l'emploi d'appareils à codage digital des hauteurs d'eau sur bandes perforées (Ficher – Porter ; OTT).

Les ingénieurs d'Electricité de France, comme la plupart de leurs collègues européens, souhaitaient conserver les limnigraphes à côté des codeurs numériques. Il semble, en effet, que l'enregistrement analogique soit irremplaçable du point de vue qualitatif, car il permet d'un seul coup d'œil sur le terrain un jugement sur le fonctionnement de la station, la qualité de l'enregistrement et l'allure des phénomènes. D'un point de vue quantitatif même, il est précieux pour déterminer *in situ* certaines valeurs telles que l'heure et la hauteur d'une pointe de crue.

Après étude de diverses solutions, le système retenu est celui de l'adjonction aux enregistreurs en place d'un dispositif codeur-enregistreur sur bande magnétique.

La solution du codage sur bande magnétique a été adoptée, comme l'ont également fait certains hydrologues au Royaume-Uni, parce qu'il a paru après étude qu'un appareil de ce type serait moins onéreux, de plus faible volume et exigerait une source d'énergie moins importante qu'un codeur à bande perforée.

1. Geological Survey des Etats-Unis d'Amérique, Ministère des richesses naturelles du Québec, etc.

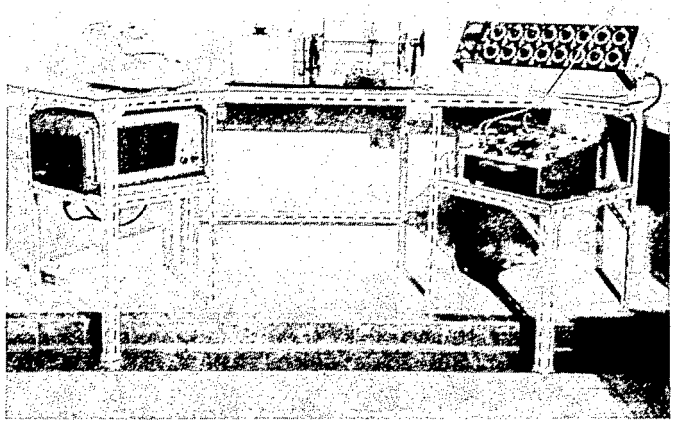


Figure 1 – *Analyseur de limnigrammes, 1961*

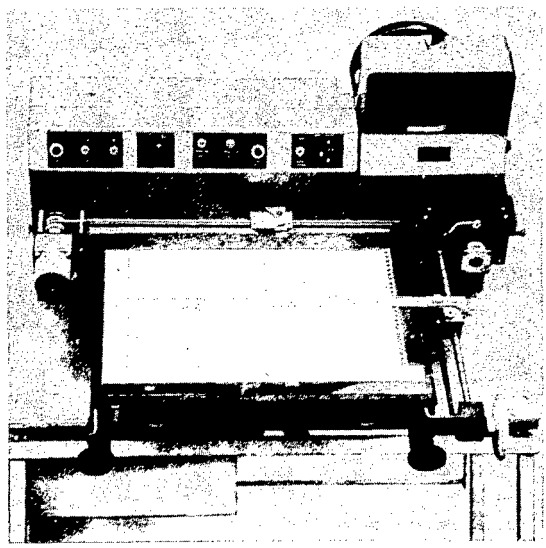


Figure 2 – *Analyseur de limnigrammes – perforateur, 1968*

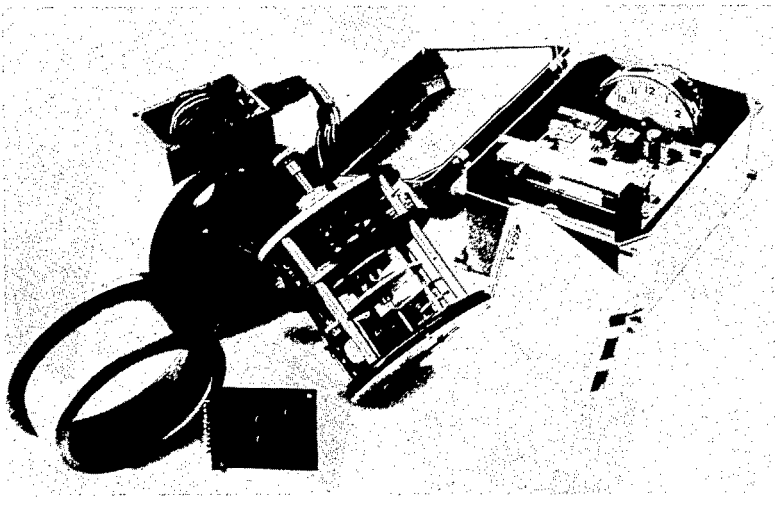


Figure 3 – *Ensemble codeur-enregistreur magnétique incrémental pour limnigraphe*

La réalisation des premiers prototypes permet d'espérer qu'un enregistreur coûterait 500 francs (pour la mesure de la pluie), qu'un codeur-enregistreur (pour la mesure des hauteurs d'eau) serait également d'un coût réduit et qu'il serait possible avec des piles classiques d'assurer une autonomie d'un an avec un enregistrement toutes les cinq minutes sur une bande de 25 mètres de longueur. Les bandes magnétiques se présentent en cassettes du commerce, qu'on peut expédier par la poste.

Un dispositif spécial à grande vitesse de lecture, d'un prix réduit, permettra de transformer les bandes magnétiques des cassettes en bandes magnétiques assimilables par calculateur, après introduction sur un clavier des données d'identification.

Ces nouvelles bandes seront traitées ensuite sur calculateur pour assurer la transformation des relevés de hauteurs en valeurs de débits : le reste du traitement sera analogue à ce qu'il est actuellement.

Une première série de ces appareils subit actuellement des essais complets tant sur le terrain qu'en laboratoire. Les résultats sont encourageants et leur mise en exploitation définitive est prévisible dans un délai relativement court.

CONCLUSION

Le traitement mécanographique des données permettra un gain de temps plus appréciable lorsque les stations seront toutes munies de codeurs numériques sur le terrain, mais l'expérience de services tels que le Geological Survey des Etats-Unis montre que l'avantage du système réside essentiellement dans la rationalisation du travail, l'amélioration de la qualité des résultats et la possibilité d'utiliser ultérieurement, pour toute étude hydrologique, les données archivées sur bande ou sur disque magnétique au fur et à mesure de leur traitement et de leur publication.

Ce mode de traitement de l'information semble d'autant plus nécessaire que les débits naturels sont de plus en plus rares et qu'il est souvent nécessaire de procéder à

des calculs de reconstitution des débits naturels mettant en jeu un grand nombre de données, dont certaines ont des répercussions sur de nombreuses autres.

Toutefois, il paraît indispensable de rechercher activement une méthode de contrôle permanent du débit permettant d'obtenir directement sur le terrain les valeurs du débit en système numérique codé afin d'économiser les jaugeages nécessaires à l'établissement de la courbe hauteur-débit et de supprimer cette variable intermédiaire qu'est la hauteur d'eau.

DISCUSSION

Rossi. Je voudrais ajouter à ce qu'a dit M. Douillet à propos des magnétophones que nous avons déjà installé deux appareils d'enregistrement pour enregistrer la pluie en différents points de mesure : dans les Alpes (c'est-à-dire à 2 700 mètres au-dessus du niveau de la mer), au niveau de la mer, tout près de Venise et dans une sous-station électrique, pour contrôler le bon fonctionnement de l'enregistreur.

Nous en avons installé un autre en Italie, ainsi qu'un appareil pour le contrôle des variations des niveaux d'eau sur un maréographe à Venise. Le fonctionnement de tous les appareils était satisfaisant.

Errors in measurement of flow by velocity area methods

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Abstract. Routine measurements of flow in open channels, carried out using current meters and applying the velocity area method, do not give sufficient information to estimate the accuracy of the method. In order to determine the magnitude of the errors much more detailed measurements are needed. This paper deals with the analysis of these detailed measurements.

Attention has been paid to errors due to:

1. Using a finite time to measure the local point velocities;
2. Using a finite number of points per vertical;
3. Using a finite number of verticals.

Statistical methods have been used extensively in the analysis. Results of the analysis of measurements in the River Yssel will be presented. This work is part of a project for the International Organization for Standardization.

ERREURS DANS LA MESURE DES DEBITS PAR LA METHODE DES ISOCHROMES

Résumé. Les mesures courantes d'un écoulement dans des canaux découverts, effectuées à l'aide d'un moulinet par la méthode de l'intégration du champ des vitesses, ne donnent pas d'informations suffisantes pour estimer la précision de la méthode. Pour déterminer la grandeur des erreurs on a besoin de mesures plus détaillées. L'article décrit la méthode d'analyse de ces mesures plus détaillées.

On étudie avec soin l'analyse des erreurs dues à : (a) l'emploi d'un temps limité pour la mesure de vitesses en un point ; (b) l'emploi d'un nombre limité de points par verticale ; (c) l'emploi d'un nombre limité de verticales.

Un usage considérable des méthodes statistiques a été fait dans les analyses. On donne les résultats des analyses de mesures dans le fleuve Yssel. Ce travail fait partie d'un projet de l'Organisation internationale de normalisation.

ERRORES EN LA MEDICIÓN DE CAUDAL POR MÉTODOS DE ÁREA VELOCIDAD

Resumen. Las mediciones de rutina de caudal en cauces abiertos, realizadas utilizando medidores de corriente y aplicando el método de área velocidad, no dan información suficiente para estimar la precisión del método. A fin de determinar la magnitud de los errores se precisan mediciones mucho más detalladas. La comunicación trata del análisis de estas mediciones y presta particular atención a los errores debidos a:

1. Uso de un tiempo finito para medir velocidades puntuales locales.
2. Uso de un número finito de puntos por vertical, y
3. Uso de un número finito de verticales.

En el análisis se han utilizado profusamente métodos estadísticos y se presentarán los resultados del análisis de mediciones en el río Yssel. Este trabajo es parte de un proyecto para la Organización Internacional de Normalización.

ОШИБКИ В ИЗМЕРЕНИИ РАСКОДА МЕТОДОМ « СКОРОСТЬ-ПЛОЩАДЬ »

Резюме. Стандартные измерения расхода в открытых руслах, выполняемые вертушками и применением метода « скорость-площадь », не дают достаточной информации, чтобы оценить точность этого метода. Для определения величины ошибки требуется гораздо больше детальных измерений. В статье проведен анализ этих детальных измерений. Внимание было привлечено к ошибкам, обусловленным :

1. Использованием ограниченного времени для измерения скоростей в точках,
2. Использованием определенного числа точек на вертикали и
3. Использованием определенного числа вертикалей.

Успешно были использованы статистические методы в этом анализе. Излагаются результаты анализа измерений на р. Иссель. Данная работа является частью работы для Международной организации по стандартизации.

NOTATIONS

b	width of channel
b_i	width of ith vertical
d_i	depth of ith vertical
E	ensemble average or mathematical expectation
$h(x)$	depth profile
m	number of verticals in a cross-section
p	number of points in a vertical
q	unknown discharge
Q	calculated discharge
Q_i	calculated discharge of ith segment
S	sampling error
t	time
t_0	duration of averaging instantaneous velocity
U_i	measured velocity
$v(x, y)$	velocity field over width and depth
v_{ik}	true velocity in point k in vertical i
$\overline{v_i}$	true mean velocity in vertical i
$V(t)$	instantaneous local point velocity
V_{ik}	measured velocity in point k in vertical i
$\overline{V_i}$	calculated mean velocity in vertical i
x	distance from one of the banks
X	measured value
y	distance from water surface
α_k	weight of velocity in point k
ϵ_j'	relative root mean squared error of error j
λ	inverse of eulerian integral time scale
μ_*	mean of *
μ_j'	relative mean of error j
ξ	true value
ρ_*	autocorrelation function of *
σ_*	standard deviation of *
σ_j'	relative standard deviation of error j
τ	time displacement.

(Variables with and without errors are denoted by upper case and lower case symbols respectively.)

INTRODUCTION

The unknown discharge q is the integral of a velocity field over a cross-section (Fig. 1).

$$q = \int_0^b \int_0^{h(x)} v(x, y)\, dx\, dy. \tag{1}$$

The use of the velocity area method means that the integral is approximated by finite summations. The mean velocity in a vertical i is approximated by the sum of p local point velocities $V'_{i,k}$, each with a weight α_k.

$$\bar{V}_i = \sum_{k=1}^{p} \alpha_k \cdot V_{i,k}. \tag{2}$$

The discharge q is approximated by a sum of partial flows, products of the width b_i, depths d_i and mean velocities $\bar{V}_i$.

$$Q = \sum_{i=1}^{m} b_i d_i \bar{V}_i. \tag{3}$$

Only errors which are characteristic of the velocity area method will be investigated. These errors are due to:

1. Sampling local point velocities in time, i.e. using a finite measuring time (error 1).
2. Sampling the vertical velocity profile, i.e. using a finite number of points per vertical (error 2).
3. Sampling the horizontal velocity profile and the depth profile, i.e. using a finite number of verticals (error 3).

Quantities relating to the errors will have a corresponding subscript. The errors are supposed to be random, so in general the measured value X is the sum of the true value ξ and a random sampling error S.

$$X = \xi + S. \tag{4}$$

S can be described by its relative mean μ' —the systematic part of the error— and its relative standard deviation σ', which are defined as the following over-all averages:

$$\mu' = E \cdot \{S\}/x \tag{5}$$

$$\sigma'^2 = E \cdot \{S - E\{S\}\}^2/x^2. \tag{6}$$

A yard-stick for both the systematic and the variable parts of the error is the relative root mean squared error ϵ'.

$$\epsilon'^2 = \mu'^2 + \sigma'^2. \tag{7}$$

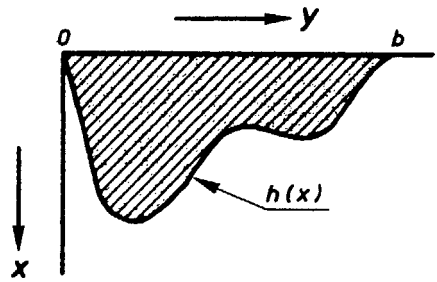


Figure 1 – *Discharge as an integral of a velocity field over a cross-section*

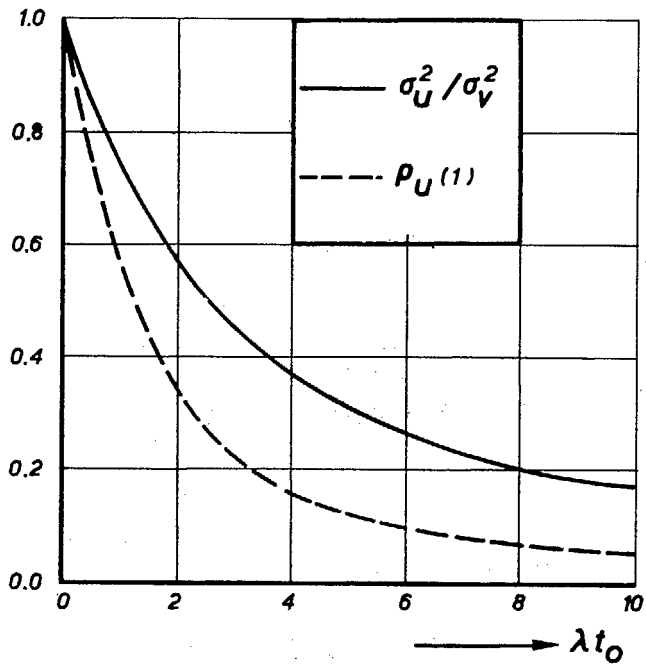


Figure 2 – *Graph for deviation σ*

The analysis of the errors is carried out by the Netherlands Technical Committee for the International Standardization Organization. Results are applied for measurements in the River Yssel in the Netherlands.

A somewhat similar investigation in this field has been carried out by Carter and Anderson (1963).

SAMPLING LOCAL POINT VELOCITIES (ERROR 1)

Model

Even for steady flow, a local point velocity will fluctuate as the result of turbulence. Hence a measurement with a current meter over a short period yields a velocity which differs from the true mean velocity. This error has been called the fluctuation error. It is assumed that it has no systematic part, so $\mu_1' = 0$.

The instantaneous local point velocity $V(t)$ can be regarded as a stochastic process with time t as index. It will be assumed that this process is gaussian, i.e. for each finite number of times t_i the corresponding random variables $V(t_1)$ have a multi-dimensional normal distribution, stationary, i.e. parameters are not a function of time, and finally ergodic, i.e. over-all averages are equal to time averages. Hence the mean μ_V, variance σ_V^2 and the autocorrelation function $\rho_V(\tau)$ describe the process completely. They are defined as the following over-all averages.

$$\mu_V = E\{V(t)\} \tag{8}$$

$$\sigma_V^2 = E\{[V(t) - \mu_V]^2\} \tag{9}$$

$$\rho_V(\tau) = E\{[V(t) - \mu_V]\,[V(t+\tau) - \mu_V]\}/\sigma_V^2. \tag{10}$$

Measuring with a current meter means the averaging of $V(t)$ over a certain time t_0. So consecutively measured velocities U_i are linked with $V(t)$ by

$$U_i = \frac{1}{t_0} \cdot \int_{(i-1)\cdot t_0}^{i \cdot t_0} V(t) \cdot \mathrm{d}t; \quad i = 1, 2, 3, \ldots \tag{11}$$

It can be derived that the parameters of the time series U_i, (mean, variance and autocorrelation function) are the following functions of the parameters of $V(t)$:

$$\mu_U = \mu_V \tag{12}$$

$$\sigma_U^2 = \sigma_V^2 \cdot \frac{2}{t_0} \cdot \int_0^{t_0} \left(1 - \frac{\tau}{t_0}\right) \cdot \rho_V(\tau)\, \mathrm{d}\tau \tag{13}$$

$$\rho_U(k) = \frac{\dfrac{1}{t_0} \cdot \displaystyle\int_0^{t_0} \left(1 - \frac{\tau}{t_0}\right) \cdot [\rho_V(k \cdot t_0 + \tau) + \rho_V(k \cdot t_0 - \tau)]\, \mathrm{d}\tau}{\dfrac{2}{t_0} \cdot \displaystyle\int_0^{t_0} \left(1 - \frac{\tau}{t_0}\right) \cdot \rho_V(\tau) \cdot \mathrm{d}\tau} \tag{14}$$

$$k = 1, 2, 3, \ldots$$

These formulae can only be applied if $\rho_V(\tau)$ is known. A useful approximation is an exponential function, e.g. Hinze (1957).

$$\rho_V(\tau) = \exp(-\lambda \cdot |\tau|) \tag{15}$$

in which λ represents the inverse of the eulerian integral time scale.

Substitution of equation 15 into equations 13 and 14 yields (see Figs. 2 and 3)

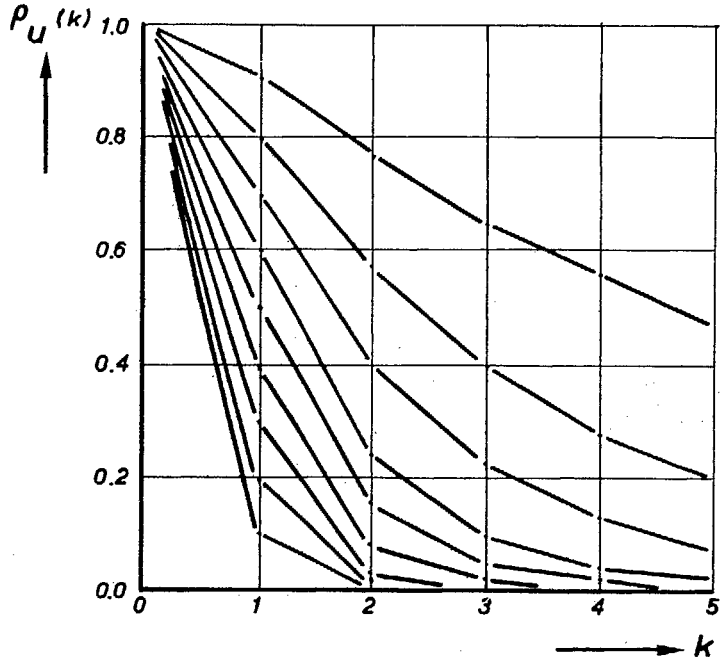


Figure 3 – *Function ρ(k)*

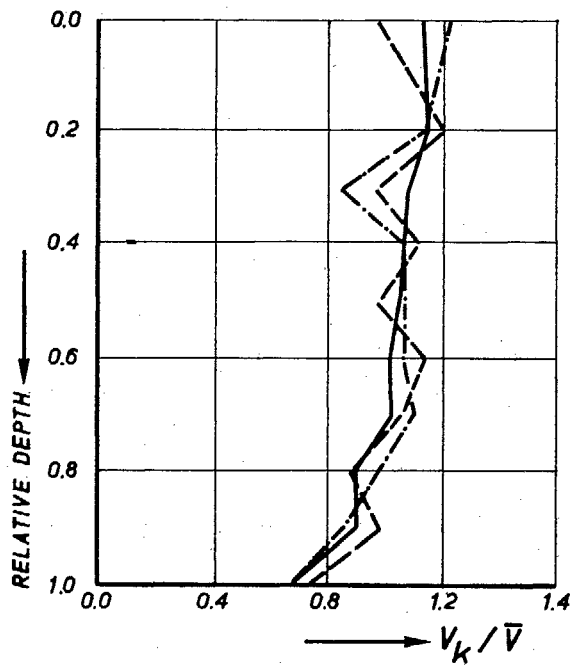


Figure 4 – *Some measured velocity profiles*

$$\frac{\sigma_U^2}{\sigma_V^2} = 2\frac{\lambda \cdot t_0 - 1 + e^{-\lambda \cdot t_0}}{(\lambda \cdot t_0)^2} \tag{16}$$

$$\rho_U(k) = \frac{1}{2}\exp\left[-(k-1)\cdot\lambda\cdot t_0\right]\cdot\frac{1-2e^{-\lambda\cdot t_0}+e^{-2\lambda\cdot t_0}}{\lambda\cdot t_0 - 1 + e^{-\lambda\cdot t_0}}\cdot \tag{17}$$

From a measurement of a series of velocities U_i, with a certain t_0, it is possible to estimate the standard deviation and the autocorrelation function. Now there are two possibilities:

1. The estimated autocorrelation function agrees with the theoretical formula, equation 17. Then λ can be calculated with equation 17 and σ_V with equation 16. Hence σ_U can be determined for an arbitrary t_0. However, t_0 should not be taken as too short an interval for the autocorrelation function (equation 15) does not hold when τ is small compared with $1/\lambda$.
2. The estimated autocorrelation function does not agree with the theoretical formula (equation 17). Then σ_U can be determined only for times longer than t_0. For a duration of measurement of $n \cdot t_0$, it can be shown that the standard deviation becomes

$$\sigma_U(n\cdot t_0) = \sigma_U(t_0)\cdot\frac{1}{\sqrt{n}}\cdot\sqrt{1+2\cdot\sum_{k=1}^{n}\left(1-\frac{k}{n}\right)\cdot\rho_U(k)}. \tag{18}$$

Measurements

Local point velocities have been measured uninterruptedly during 3,000 sec with the current meter being read every 30 sec. This has been done at 10 points in 3 verticals in a cross-section (verticals with 1; 0.6 and 0.3 times the maximum depth). The whole procedure was repeated for four different discharges. Afterwards, a further measurement was added: 2,000 sec long, readings every 10 sec, in the above-mentioned verticals, but at fewer points. The result of a measurement can be regarded as a realisation of the time series U_i.

Analysis

After removing considerable linear trend in the series, due to unsteady flow, the relative standard deviation σ_1 and the autocorrelation function $\rho_U(k)$ were estimated. The normal distribution of the velocities was investigated with the χ^2 test, after a correction for the correlation between the velocities had been made.

Results

Table 1 gives some parameters of the cross-section depicted in Figure 5. Average results of the measurements with $t_0 = 30$ sec and $t_0 = 10$ sec are summarized respectively in Tables 2 and 3. Neither result is contrary to the exponential autocorrelation function (equation 15). Predicting σ_1' for 30 sec from the results for 10 sec and vice versa gives good agreement with the model. Out of 143 analysed series, the hypothesis that the measured velocities are normally distributed had to be rejected eight times (risk of 1 per cent).

SAMPLING VERTICAL VELOCITY PROFILE (ERROR 2)

Model

The true mean velocity in a vertical $\bar{v}$ is approximated by $\bar{V}$, the sum of p local point velocities V_k with weights α_k.

$$\bar{V} = \sum_{k=1}^{p} \alpha_k \cdot V_k. \tag{19}$$

This can be regarded as sampling of the vertical velocity profile, resulting in a sampling error S_2. With respect to an arbitrary velocity profile, this error has a random character. $\bar{V}$ is also influenced by the fluctuations of the measured local point velocities, resulting in a fluctuation error S_1.

$$\bar{V} = \bar{v} + S_1 + S_2. \tag{20}$$

Both S_1 and S_2 are dependent on the choice of the local points; S_1 is also dependent on the duration of measurement. To separate the influence of the independent errors S_1 and S_2 repeated measurements of different velocity profiles are needed.

Measurements

Local point velocities have been measured consecutively for 60 sec at 10 points (near the surface and at relative depths of 0.2; 0.3; 0.5; 0.6; 0.7; 0.8; 0.9 nearer the bottom) in the deepest vertical of a cross-section. This was done 10 times without interruption. The total procedure was repeated for four different discharges. Mean velocities have been obtained with a planimeter. Some measured velocity profiles are shown in Figure 4.

The results of these measurements can be summarized as V_{ijk}; i denotes different discharges, j repetitions at the same discharge, k local points in the vertical ($i = 1, 2, \ldots, I$; $j = 1, 2, \ldots, J$).

Analysis

It is supposed that the error in the mean velocity obtained with the planimeter can be disregarded. Measured velocities have been standardized by dividing by the corresponding planimeter velocity. Then all true mean velocities are unity and deviations from it are relative. The sampling error S_2 was assumed to have been a constant during the ten repeated measurements.

Then

$$V_{i,j} = 1 + S_{1,i,j} + S_{2,i}. \tag{21}$$

The mean of the sampling error S_2 can be estimated by

$$\mu_2' = \frac{1}{I \cdot J} \cdot \sum_{i=1}^{I} \sum_{j=1}^{J} \bar{V}_{i,j} - 1. \tag{22}$$

Table 1. *Some parameters of the cross-section depicted in Figure 5*

Measurement		1	2	3	4	5
Discharge	m^3/s	550	275	160	385	650
Area cross-section	m^2	605	380	240	470	700
Mean velocity	m/s	0.91	0.72	0.67	0.82	0.93
Maximal depth	m	8.6	6.8	5.8	7.7	9.5
Width	m	108	98	86	110	110
Mean depth	m	5.6	3.9	2.8	4.7	6.4

Table 2. *Average results of measurements,* $t_0 = 30$ sec

Relative depth	"0.0"	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	"1.0"	mean
σ_1' %	4.3	3.9	4.5	4.9	5.1	6.2	7.0	8.0	9.9	10.5	6.4
$\rho_U(1)$	0.09	0.06	0.04	0.10	0.07	0.09	0.16	0.09	0.09	0.12	0.09

Table 3. *Average results of measurements,* $t_0 = 10$ sec

Relative depth	0.2	0.6	0.8	0.9	mean
σ_1' %	6.2	9.6	11.2	13.6	10.1
$\rho_U(1)$	0.27	0.30	0.33	0.25	0.28
$\rho_U(2)$	0.00	0.12	0.12	0.04	0.07

The relative standard deviation of S_2, however, is only a part of the standard deviation of $\overline{V}_{i,j}$, the other part is due to the fluctuation error S_1. It can be estimated by

$$\sigma_2' = \frac{1}{I-1} \cdot \sum_{i=1}^{I} (\overline{V}_i - 1 - \mu_2')^2 - \frac{1}{I \cdot J(J-1)} \cdot \sum_{i=1}^{I} (\overline{V}_{i,j} - \overline{V}_i)^2 \qquad (23)$$

in which

$$V_i = J^{-1} \cdot \sum_{j=1}^{J} \cdot \overline{V}_{i,j}.$$

Results

Several rules to calculate the mean velocities have been examined (Table 4). The standard deviation due to fluctuation and sampling error, σ_{1+2}', is added to illustrate the effect of the separation. It is remarkable that the two-points rule is the best and that most means are significantly different from zero.

SAMPLING HORIZONTAL VELOCITY PROFILE AND DEPTH PROFILE (ERROR 3)

Model

The true discharge q is approximated by the sum of partial flow Q_i, derived from depths, mean velocities and positions of a finite number of verticals. This implies a sampling error S_3.

$$Q = \sum_{i=1}^{n} Q_i = q + S_3. \qquad (24)$$

This error is dependent not only on the number of verticals and their positions, but also on the regularity of horizontal velocity profile and depth profile.

Measurements

A cross-section has been covered with 45 verticals at equal distances. In each vertical the depth and the velocity were measured at a relative depth of 0.6 for 120 sec. A reference velocity at one fixed point was measured to correct for unsteady flow. The procedure was carried out for four different discharges.

Analysis

The error of the discharge determined by means of all verticals was supposed to be zero. Then the discharge was calculated using fewer verticals, and the relative deviations of the first discharge were analysed. The question arose as to which

Table 4. *Examination of several rules for calculating mean velocities*

	Rules	μ_2' %	σ_2' %	ε_2' %	σ_{1+2}' %
1	$v_{0.6}$	3.6*	1.2	3.8	6.5
2	$0.96\ v_{0.5}$	2.9	3.0	4.1	6.5
3	$0.5(v_{0.2} + v_{0.8})$	0.2*	0.0	0.2	4.7
4	$1/3(v_{0.2} + v_{0.6} + v_{0.8})$	1.3*	0.0	1.3	2.6
5	$0.25(v_{0.2} + 2v_{0.6} + v_{0.8})$	1.9*	0.3	1.9	3.0
6	$0.25(v_{0.2} + v_{0.4} + v_{0.7} + v_{0.9})$	-1.7	2.0	2.6	3.0
7	$0.1(v_{0.0} + 3v_{0.2} + 3v_{0.6} + 2v_{0.8} + v_{1.0})$	-0.9*	0.0	0.9	2.3
8	$1/6(v_{0.0} + v_{0.2} + v_{0.4} + v_{0.6} + v_{0.8} + v_{1.0})$	-3.6*	0.5	3.7	2.0
9	$0.1(v_{0.0} + 2v_{0.2} + 2v_{0.4} + 2v_{0.6} + 2v_{0.8} + v_{1.0})$	-0.8*	0.0	0.8	1.8

* significant $\neq 0$

Table 5. *Results of sampling horizontal velocity profile and depth profile*

A				B				C			
m	μ_3' %	σ_3' %	ε_3' %	m	μ_3' %	σ_3' %	ε_3' %	m	μ_3' %	σ_3' %	ε_3' %
4	-14.3	3.5	14.7	4	-8.8	6.1	10.7	3	-4.0	5.4	6.7
7	- 4.5	1.2	5.9	9	-4.0	3.0	5.0	8	-3.4	3.4	4.8
12	- 1.4	1.1	1.8	18	-0.6	1.2	1.4	17	-2.3	0.8	2.4
23	0.0	2.8	2.8	27	0.0	1.3	1.3	22	-1.8	0.2	1.8

verticals should be removed, and three ways of reducing the number of verticals were adopted:

A. Keeping the distances between the verticals equal;
B. According to the judgement of an experienced hydrographer who chooses his verticals taking the depth profile and the distances between the verticals into consideration; so the depth profile has to be known in advance;
C. Keeping the partial flows as nearly equal as possible. This can be done, however, only when some previous knowledge of horizontal velocity and depth profiles is available.

Discharges were computed by the mid-section method, i.e. the product of depth and velocity is assumed to vary linearly with the distance between consecutive verticals.

Results

Results are summarized in Table 5 for methods A, B and C. Examples of thinning out patterms, according to methods B and C, are shown in Figure 5.

A surprising fact is that the mean errors μ_3^1 is always negative. The three methods of choosing verticals do not give very different results. But for very few verticals only, it can be seen that the more information used in choosing the verticals, the better the result.

COMBINATION OF ERRORS

Supposing the three errors have a mean zero and are independent, it can be derived that they result in an error of discharge with a standard deviation σ_t

$$\sigma_t^2 = \sigma_3'^2 + \sum_{i=1}^{m} \frac{(b_i \cdot d_i \cdot v_i)^2}{q} \cdot \left[\sigma_2'^2 + \sum_{k=1}^{P} \alpha_k^2 \cdot \left(\frac{v_{i,k}}{\bar{v}_i}\right)^2 \cdot \sigma_{i,k}'^2\right]. \quad (25)$$

Use of this formula, with $b_i d_i \bar{v}_i / q = 1/\text{m}$, taking values of ϵ' instead of σ' to be on the safe side, and using ϵ_3' according to method B, has led to Figure 6. The number of verticals appears to have the most influence, which is not surprising if the form of the cross-section is taken into account.

CONCLUSIONS

Regarding Error 1 the following conclusions can be drawn: σ_1' increases with increasing depth; no influence of the position of the vertical, discharge or other flow parameters on σ_1' has been found; the theoretical autocorrelation function $\exp[-\lambda \cdot |\tau|]$ has not been rejected; and velocities measured with a current meter during 30, and even 10, can be considered to be normally distributed.

Regarding Errors 2 and 3 it should be realized that the results are based on measurements in one cross-section at four different discharges. To draw general conclusions more measurements are needed, and these have become available as a result of the ISO investigations.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Carter, R. W.; Anderson, I. E. 1963. Accuracy of current meter measurements. *Proceedings A.S.C.E. Hy*, vol. 4, July.
2. Hinze, J. O. 1951. *Turbulence.* New-York, McGraw-Hill 1957.

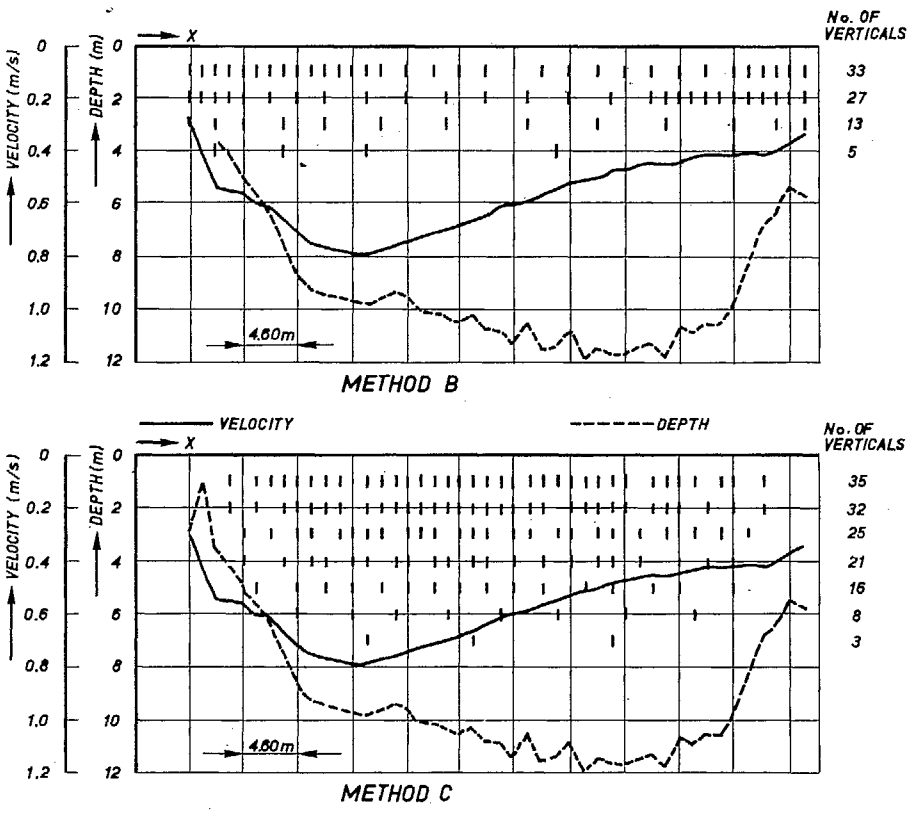


Figure 5 – *Patterns of velocity, depth and distance from the banks*

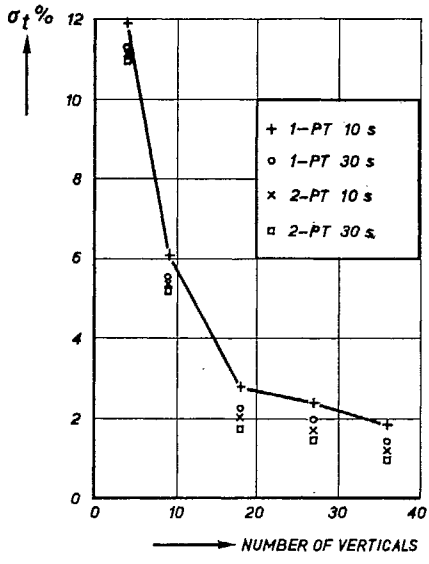


Figure 6 – *Decrease of deviation with the number of verticals*

DISCUSSION

Lambie: Pulsating or fluctuating velocities may be caused by secondary currents due to the hydraulic conditions. Did you try to correlate the physical characteristics of the site with the hydraulic conditions in the approached channel and their effect on the pulsation itself?

Botma: We tried to correlate our results with the primacies of the cross-section, but we did not find any significant results. In this case we only used one cross-section under different circumstances.

Lambie: A second question: When sampling local point velocities at ten points in each vertical, were the samples taken by ten meters simultaneously, or one at a time by the same meter? If simultaneous-readings were not taken, was an attempt made to take simultaneous reading at the more commonly used points, such as 0.2, 0.6 or 0.8?

Botma: In the investigations of the first error, we did measure simultaneously at different points in the vertical.

Lambie: A third question: At each site, has it yet been possible to detect the length of any short-term or long-term cycles in the pulsation period? And if so, has any conclusion been drawn from this on the optimum observation time?

Botma: We did not find any harmonic pulsation.

Hinrich: According to the rules, for the measurement of discharges in the Federal Republic of Germany each measurement shall last 30 sec. Do you think that 10 sec would be sufficient according to your observations ?

Botma: When using only two points, the exposure time should not be too short. If you are taking more points in a vertical, then the number of points averages the pulsation and you get a better result. If you are going from 30 to 10 sec then I think that it is not enough for the two-point method. With other methods at more points in a vertical, it may be enough.

Hinrich: We do apply three different methods with the current meter: measurement at many points; measurement at two points; integrating measurement with a velocity of decline of 2 cm/sec. The accuracy is generally about ± 4 per cent and in many cases even ± 1 to 2 per cent.

Botma: There are perhaps better methods to measure the velocity in a vertical. But you should realize that we had already taken the data, or the data were given to us by the ISO. Perhaps it would be very interesting to compare all these methods, but that was not the goal of the ISO investigations.

The criterion of information sufficiency with automation of hydrological measurements

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Abstract. Hydrological investigations necessitate the study of a continuous process using a certain series of discrete values. Moreover, the amount of information required to represent the process to a required accuracy must be determined. The method considered assesses the optimum number of measurements by an observer or by a recorder for a given accuracy. Error distribution curves testify that the accuracy and the frequency of measurements are related by an exponential function. On the basis of these curves, the efficiency of hydrometric measurements may be determined, and a superfluous complexity of automatic measuring devices may be avoided.

CRITERE D'UNE INFORMATION SUFFISANTE DANS L'AUTOMATISATION DES MESURES HYDROLOGIQUES

Résumé. Dans les recherches hydrologiques la nécessité apparaît d'étudier un processus continu utilisant certaines séries de valeurs discontinues. De plus, on doit résoudre le problème d'une information suffisante pour représenter le processus étudié avec la précision requise. La méthode considérée fournit une détermination facile du nombre optimal de mesures que doivent effectuer l'observateur ou l'enregistreur pour restituer avec une précision donnée la fonction présumée. Les courbes de distribution des erreurs montrent que la précision et la fréquence des mesures sont reliées par une fonction exponentielle.

Sur la base de ces courbes, il est possible en outre de déterminer la valeur des mesures hydrométriques et d'éviter de façon importante une complexité superflue des mécanismes de mesure automatique.

CRITERIO DE SUFICIENCIA DE INFORMACION EN LA AUTOMATIZACIÓN DE MEDICIONES HIDROLÓGICAS

Resumen. Al realizar investigaciones hidrológicas surge la necesidad de estudiar un proceso continuo utilizando una serie determinada de valores discretos. Se plantea el problema sobre la suficiencia de información para representar el proceso que se estudia con la precisión necesaria. El método considerado permite una determinación fácil del número óptimo de mediciones por observador o por registrador y la precisión necesaria. Las curvas de distribución de error testifican que la precisión y la frecuencia de las mediciones están relacionadas por una función exponencial. Basándose en dichas curvas, se puede determinar además la garantía de las mediciones hidrométricas y se puede evitar en forma considerable una complejidad superflua de los dispositivos automáticos de medición.

КРИТЕРИЙ ДОСТАТОЧНОСТИ ИНФРМАЦИИ ПРИ АВТОМАТИЗАЦИИ ПРОЦЕССОВ ГИДРОЛОГИЧЕСКИХ ИЗМЕРЕНИЙ

Резюме. При гидрологических исследованиях возникает необходимость изучить процесс, непрерывный по своей природе, с помощью некоторого ряда дискретных значений. При этом требуется решить задачу о достаточном количестве информации, чтобы изучаемый процесс был представлен с заданной точностью. Рассматриваемый метод позволяет при заданной точности восстановления исходной функции сравнительно легко определить оптимальное количество измерений наблюдателем или автоматом. Кривые распределения погрешностей показывают, что точность и частота измерений связана показательной функцией. На основании этих кривых, кроме того, может быть определена эффективность гидрометрических работ и, в значительной мере, предотвращена излишняя сложность измерительных автоматов.

In formation theory, the problem of transmission of information is solved, for example, by means of frequency signals. Kotelnikov (1933) gave a theoretical proof of the possibility of exact determination of the minimum and sufficient number of discrete electrical signals in a communication line.

The essence of Kotelnikov's theorem is the following : if the time funciton $f(t)$ is preset and its spectrum is limited by a frequency band from 0 to ω_c, it is necessary to define the maximum allowable interval between impulses which will guarantee the reproduction of function $f(t)$ to a specified accuracy.

The proof of this theorem is based on the expansion of function $f(t)$ into a series according to which

$$f(t) = \sum_{n=-\infty}^{n=\infty} f(n \cdot \Delta t) \cdot \frac{\sin \omega_c (t - n \cdot \Delta t)}{\omega_c (t - n \cdot \Delta t)} \cdot \tag{1}$$

From equation 1 it is clear that function $f(t)$ is expressed through its instantaneous $f(n \cdot \Delta t)$ values, i.e. time intervals equal to

$$\Delta t = \frac{\pi}{\omega_c} = \frac{1}{2 \cdot f_c} \tag{2}$$

where f_c is the maximum frequency in the spectrum of function $f(t)$. From this it follows that if period $f(t)$ equals T, function $f(t)$ may be written:

$$m = T/\Delta t = 2 \cdot f_c \cdot T. \tag{3}$$

The criterion of information sufficiency allows any continuous process with a relatively limited frequency spectrum to be presented as a function of discrete values of the processes under study. To determine f_c the function $f(t)$ should be expanded into a Fourier series, provided it satisfies the conditions of Dirichlet.

The expansion into the series is performed by analytical or approximate methods and with the help of analysers or special stencils (Lopshitz or Smirnov). It is evident that this theorem is true not only for functions dependent on time, but also for the study of processes depending on co-ordinates. Thus, where preliminary information on the process under study (such as an initial curve or analogue) is available and it is necessary to go on with a detailed investigation on the special structure of this phenomenon or to study the nature of the process in time, then it is possible to determine accurately the amount of information necessary to reproduce the phenomenon or the process under study with the specified accuracy. This method may be used to determine the measurement frequency of almost any hydrometeorological element: bed profile, water level and discharge, temperature distribution, flow velocities, liquid and solid precipitation. In all the cases it is essential to know the number of stations or observation points, profiles or sections necessary to represent the phenomenon. In addition, the above method makes it possible to determine the frequency of transmission of discrete information by automatic hydrometeorological measuring devices or by telemetric instruments and also to design the measuring devices efficiently.

If the true nature of the process under study is known, the problem, with reference to the study of hydrometeorological elements, may be reduced to the normal law of error distribution, bearing in mind that the synthesized function is always correlated with the true function. Determination of the assumed function is of no great difficulty if a large amount of experimental data is available. When there

is no initial curve of the element under study, it is possible to use the analogue of this function in many cases. It is assumed that the initial curve for a definite time interval is stable. Small changes have no influence upon the computed results. What is important is the identity of the functions spectrum densities. When a function $f(x)$ is to be approximated by a sequence of discrete values $f(a)$, $f(a+k)$, $f(a+2k)$ . . . the optimum value of k for a given measurement accuracy may also be determined with the help of correlation theory.

However, the correlation method and the method of statistical tests deal with relationships between values, each of which varies due to random factors, and, in addition, they are more complicated.

If, for instance, it is necessary to determine the number of depth measurements in order to obtain a bed profile with adequate accuracy for discharge estimation or to determine the time of exposure of a current meter in the intergration method, it can be shown that for a complicated profile

$$H(l) = \sum_{n=0}^{n=10} \left(a_n \cdot \cos \frac{2\pi \cdot n \cdot x}{l} + b_n \cdot \sin \frac{2\pi \cdot n \cdot x}{l} \right) \tag{4}$$

where H and l are the depth and the width of the river in metres respectively.

The comparison of the $H(l)$ curve synthesized out of ten harmonics with the initial curve and error estimation show that both curves practically coincide. The value of the mean square error σ equals 0.05 m, or the mean relative error of depth measurement (average depth 2.4 m, river width 900 m) is 2 per cent.

Thus, within the limits of the above accuracy, the maximum harmonic is determined ($n = 10$) and, consequently, according to equation 3 it is possible to determine $m = 2f_c \cdot l = 20$, if the river width is assumed to be $l = 1$. Since in this example the river width is equal to 900 m, the necessary and sufficient interval between the depth measurements should be $900 : 20 = 45$ m for the error not to be greater than 2 per cent.

In the course of making discrete water level measurements in time it can be shown that, in many cases,

$$h(t) = \sum_{n=0}^{n=5} \left(a_n \cdot \cos \frac{2\pi \cdot n \cdot t}{T} + b_n \cdot \sin \frac{2\pi \cdot n \cdot t}{T} \right) \tag{5}$$

i.e. it is enough to expand $h(t)$ to four or five harmonics to obtain a satisfactory result in the case of rather complicated curves of water level variations in time. Since the initial curves $h(t)$ were obtained during 24 hr, it is necessary to perform 8-10 observations after each 3-2.4 hr, respectively, for the error to be within 2 per cent.

The comparison of synthesized curves and initial curves showed that, in the case of a sharp change of water level (rain storms, storm runoff), the relative error may amount to 6-10 per cent. When the river is in a quieter state under the same conditions ($m = 8$-10), the error may be 1 per cent or less.

It is easy to determine the relationship between error value and measurement frequency $\delta\ \% = f(m)$ and to find m on the basis of the assumed allowable error and then to estimate the time interval between measurements. By means of the same computation, the frequency of switching of an automatic measuring instrument by a programme control device is determined.

Discretization of discharge curves for the time interval T for various types of river, in order to estimate discharge with a given accuracy, is of great practical interest. Consideration and analysis of typical $Q(t)$ for ten characteristic types of rivers, after

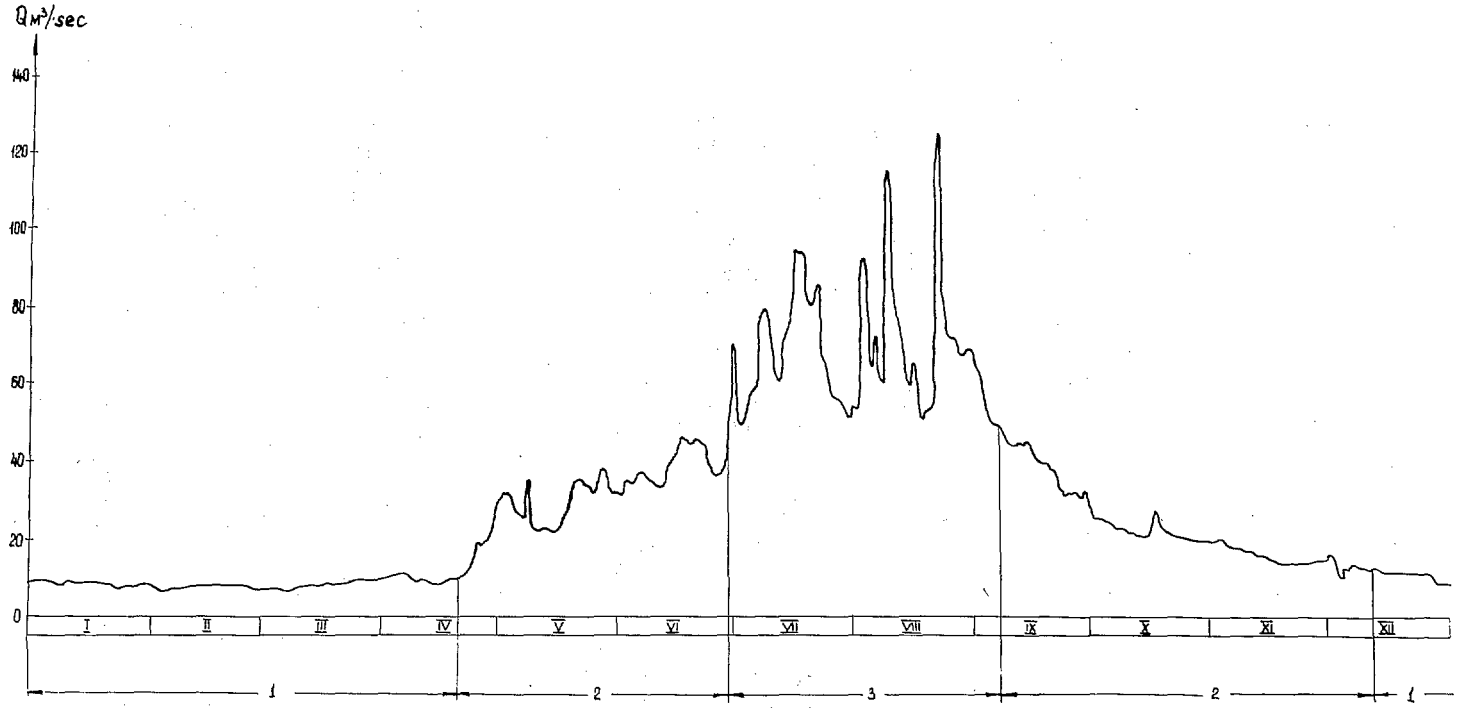


Figure 1 – *Hydrograph of the Terek river at Kazbek (Tian-Shan type). 1, 2 and 3 refer to the first, second and third groups of complexity of Q(t) respectively*

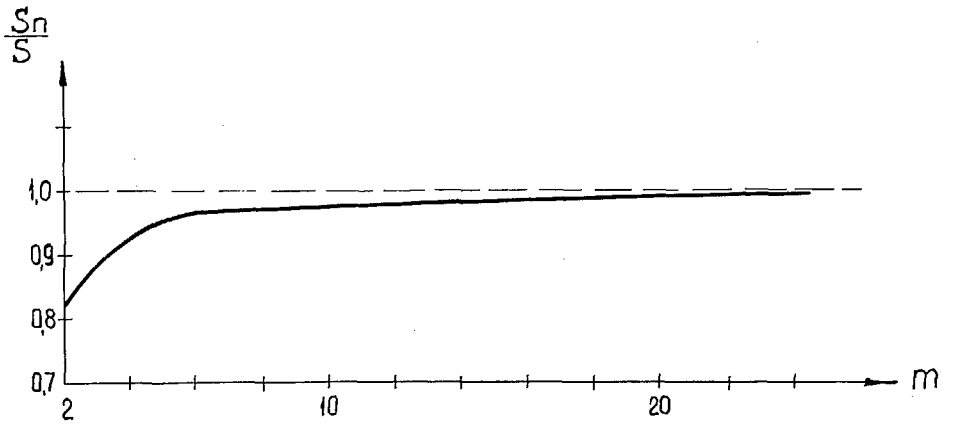


Figure 2 – *Mean ratio of the area, limited by the synthesized curve for the + nth harmonic (S_n) and by the initial curve of discharge (S) versus the frequency of measurements (m)*

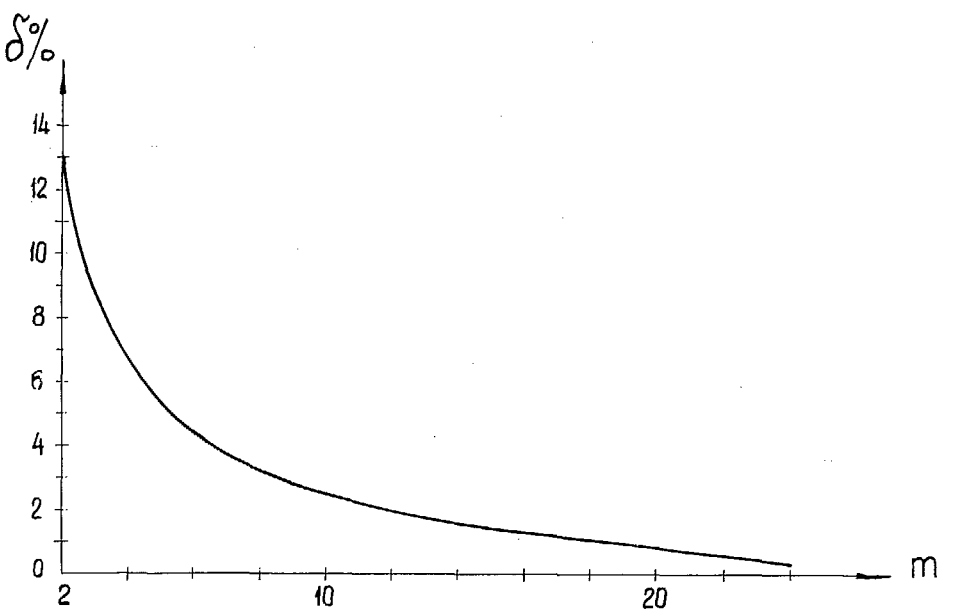


Figure 3 – *Dependence of mean errors of runoff inventories (σ %) on the frequency of measurement (m).*

Zaikov, over the U.R.S.S. territory showed that they may be subdivided into three groups according to their complexity.

The first group includes $Q(t)$ functions with smooth runoff changes in time, mainly occurring in winter and in early spring. The second group is a transitional one, when the $Q(t)$ changes become considerable (mainly spring-summer and autumn-winter periods). The third group includes periods with sharp and high flood waves, and happens late in spring, in summer and early in autumn.

For a more detailed study these groups may be subdivided into sub-groups for some river types.

Naturally, the variation in time of $Q(t)$ of some rivers in certain years (months) may differ substantially from that of characteristic rivers. For each river, however, and still more for each gauging site, $Q(t)$ changeability in time would be substantially less and, in addition, it is possible to choose a more precise analogue to ensure more accurate runoff estimation.

Finally, to determine measurement frequency at a given accuracy, from the point of view of harmonic analysis, the complexity, not the phase shift of the function in time, is of the greatest importance.

The analysis of a great number of hydrographs revealed that the frequency of discharge measurements tends to increase with an increase in complexity of $Q(t)$ thus confirming the correctness of the physical nature of the method.

In order to represent the time characteristics of $Q(t)$, a hydrograph of Tian-Shan type (the Terek river at Zazbek) is given in Figure 1, where stretches 1, 2 and 3 relate to the first, second and third order of complexity, respectively. The third order (stretch 3) in Figure 1 is of average complexity.

Let us consider the third order $Q(t)$ in relation to time, since it is of the greatest interest.

To determine the rate of discretization, the ten types of rivers have been subdivided into the forty most complex reaches and the frequency of discharge measurements for a period T has been estimated for them. The mean ratio of the area limited by the synthesized curve for the nth harmonic (S_n) to the area limited by the initial curve of discharge (S) according to the frequency of measurements m is given if Figure 2.

$S_n/S = f(m)$ is the most objective way to evaluate the restoration in time of the function $Q(t)$. As will be apparent from Figure 2, the ratio S_n/S approximates 1 (100 per cent) if m = 8-12.

It is extremely important that there is no considerable improvement of runoff values with a further increase in the frequency of measurements. The increase of m is not effective economically since the curve asymptotically approaches the actual water discharge.

Figure 3 presents the ratio of mean errors with the values of runoff from m, i.e. $\delta\ \% = f(m)$ for the same forty analysed discharges. This ratio shows that a mean error of 0.5, 1 and 5 per cent may be obtained if m equals 24, 20 and 6, respectively. It should be borne in mind that ratios $S_n/S = f(m)$ and $\delta\ \% = f(m)$ are given for mean values from a great number of analysed and synthesized discharge curves. In some regions, under different climatic conditions, certain deviations of m and $\delta\ \%$ from the mean value may be observed. These ratios, however, serve as a verification of the selection of m and establish the relationship between the accuracy of runoff determination and measurement frequency during the period T.

Since the measurement frequency m is estimated for a definite time period, it is not difficult to compute the interval of measurements using equation 3:

$$\Delta t = \frac{T}{m} \tag{6}$$

if the accuracy of runoff is given. It is quite obligatory that in equation 6 the time scale of Δt and T should be the same.

Table 1. *Results of computing mean Δt values (in days) with various errors in discharge*

Type of river	Δt			
	Error in %: up to 1	up to 5	up to 10	up to 15
Kazakhstan	< 1	3	6	12
East European	< 2	12	> 12	> 12
West Siberian	< 1	3-12	> 12	> 12
East Siberian	< 1.5	3	4.5	9
Altai	< 1	2.5	3	> 3
Far East	< 1	2	> 2	> 3
Tian-Shan	< 0.5	2	> 2	> 3
Black Sea Basin	< 1	2	> 2	> 3
Crimean	< 0.5	1	2	> 2
North Caucasian	0.3	1	1.5	2

Table 1 gives the results of computing mean Δt values in days for the forty discharge curves investigated with the errors in runoff of 1, 5, 10 and 15 per cent. During computation, only the most complex parts of hydrographs of ten river types of the U.S.S.R. have been selected on the basis of the initial data on mean annual runoff. In addition, complex parts $Q(t)$ of some other rivers have been analysed.

The comparison reveals no prominent deviation from the table. This suggests that it is quite easy to determine the criterion of sufficiency of information with a definite accuracy of runoff using a complexity grouping (spectral density) of the function in time.

The table shows that for rivers of the Kazadhstan type (for runoff with accuracy up to $\delta\,\% = 1$, 5, 10 and 15 per cent), the measurements of discharge should be performed for $t = 1, 3, 6$ and 12 days respectively (in the table Δt values are rounded off). In some cases $\Delta t = 0.3$, 0.5 days and less. It is rather difficult to arrange such frequent measurements of discharge manually. This process, therefore, should be automated.

It should be noted that for values of $Q(t)$ with a complexity similar to 2 (Fig. 1) and especially to 1, the interval Δt is much greater. The evaluation of these cases, however, is not given (they cause no major difficulties).

It may be noted that $\delta\,\%$ does not contain the errors inherent in the initial function $Q(t)$ relating to the methods and accuracies of discharge estimation. These errors, (measurements of discharge variations, related to one and the same water level), however, should be considered in more accurate computation of Δt (see Table 1). These errors may be examined by different methods, including methods of correlation analysis of random functions, which are not presented in the paper.

It should be mentioned that the method considered allows the estimation of the amount of information required during the investigation of other elements of the hydrometeorological regime. It is evident that automatic measuring devices should be

designed taking into account the criterion of information sufficiency. This method may be also applied when the data are presented as continuous curves on recorder tapes. In this case, it is possible to make an objective estimation of the minimum and sufficient amount of information for reproducing the curve being processed.

BIBLIOGRAPHY / BIBLIOGRAPHIE

1. Kotelnikov, V. A. 1933. O propusknoi sposobnosti "Efira" i provoloki v elektrosviazi.[On the conduction capacity of ether and wire in electric transmission]. *Izd. Upravlenia sviazi RKKA.*
2. Dimaksian, A. M.; Pochtarev, V. I. 1963. Primenenie nekotorykh metodov teorii informatsii pri izuchenii gidrometeorologicheskikh protsessov. [Application of some methods of theory of information at the study of hydrometeorological processes]. *J. Meteorologia i gidrologia,* no.12, 1960.
3. Chebotarev, A. I. 1960. *Obschaja gidrologia.* [General hydrology]. Gidrometeoizdat.

Dépouillement sur ordinateur des campagnes de sondage et tracé automatique des cartes bathymétriques

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Résumé. Tout sondage bathymétrique classique comprend une phase "mesures sur le terrain" et une phase "dépouillement en bureau".

L'embarcation, équipée d'un écho-sondeur, suit des routes et, à intervalles réguliers, des opérateurs mesurent, par rapport à des points connus, des angles qui permettront de déterminer la position de l'embarcation à des instants qui sont repérés sur l'enregistrement de l'écho-sondeur. On enregistre également les variations de niveau du plan d'eau (marée, par exemple), afin d'en connaître à tout instant la cote par rapport à un zéro de référence et de pouvoir rapporter chaque sonde à ce zéro.

La phase dépouillement consiste à reporter les angles mesurés de façon à définir les points repérés de la route suivie, à dépouiller les bandes d'enregistrement de l'écho-sondeur et du marégraphe et à répartir, après correction de marée, les profondeurs mesurées le long de la route suivie en utilisant les repères de temps. Cette phase est très longue et grève considérablement le coût d'une campagne de sondages lorsque toutes ces opérations sont faites à la main.

La SOGREAH utilise les possibilités d'un ordinateur, d'un lecteur de courbes et d'une table traçante pour traiter les informations recueillies et obtenir la minute de sonde et la carte bathymétrique finale avec un minimum d'opérations manuelles. On arrive ainsi à diviser par deux le coût de la phase "dépouillement" et par quatre les délais de fourniture des résultats. Les programmes de traitement sont très souples et admettent pratiquement toutes les méthodes utilisées sur le terrain pour le recueil des informations.

COMPUTERIZED DATA REDUCTION AND AUTOMATIC PLOTTING OF DEPTH CHARTS

Abstract. All conventional depth-sounding surveys include a phase of field measurements on a data-reduction phase.

The survey craft equipped with an echo-sounder makes a run and, at regular intervals, the operator measures angles with respect to known points; thus, the boat's position at those times marked on the sounding chart can be determined. The variations in the surface level of the body of the water (due to the tide, for example) are also recorded continuously with respect to a datum level, enabling each depth pick-up to be referred to this datum.

The data-reduction phase consists of plotting the measured angles in order to define the points picked out along the run, taking the data off the echo-sounder and tide recorder charts and, after correcting for tide, distributing the measured depths along the run with reference to the time base. This part of the programme is both very long and extremely costly when all these operations are done by hand.

However, a computer, a curve reader, and a plotter to process the data and to obtain the skeleton and final depth charts with a minimum of manual operations, can be used. By this means the cost of the data-reduction is cut by half, while the time required for its execution is reduced to one-fourth. The data treatment programmes are flexible and adapt to practically all the data gathering methods that are used in the field.

ANÁLISIS DE DATOS POR ORDENADOR DE CAMPAÑAS DE SONDEO Y TRAZADO AUTOMÁTICO DE PLANOS BATIMÉTRICOS

Resumen. Todos los estudios convencionales de profundidades, incluyen una fase de mediciones en campo y una fase de análisis de datos.

La embarcación equipada con un ecosonda da una pasada, y a intervalos regulares, el operador mide ángulos con respecto a puntos conocidos ; de esta forma, se puede determinar la posición de la barca en dichos momentos marcada en la carta del sondador. Las variaciones en el nivel de la superficie del cuerpo de agua (debidas a la marea, por ejemplo) se registran también en forma continua respecto a un nivel dado, permitiendo referir cada toma de profundidad a dicho dato.

La fase de reducción de datos consiste en representar los ángulos medidos a fin de definir los puntos que se toman en la pasada, a partir de los datos del ecosonda y de las cartas registradoras de mareas, después de corregir los efectos de esta última, distribuyendo las profundidades medidas a lo largo de la pasada con respecto al tiempo base. Esta parte del programa es muy larga y extremadamente costosa cuando todas estas operaciones se realizan a mano.

La SOGREAH utiliza un ordenador, un lector de curva y un trazador para procesar los datos y obtener el esquema de los planos batimétricos y el plano final con un mínimo de operaciones manuales.

Con estos medios, el costo de la reducción de datos se reduce a la mitad, mientras que el tiempo necesario para su ejecución se reduce a la cuarta parte. Los programas de tratamiento de datos son flexibles y se adaptan a prácticamente todos los métodos para la recogida de datos de campo.

ОБРАБОТКА ДАННЫХ С ПОМОЩЬЮ ЭВМ И АВТОМАТИЧЕСКОЕ ПОЛУЧЕНИЕ КАРТЫ ГЛУБИН

Резюме. Все обычные съемки глубин включают полевые измерения и уменьшение данных при составлении плана.

Судно, предназначенное для съемки, оборудованное эхолотом, начинает двигаться, и через правильные интервалы времени оператор измеряет углы по отношению к известным точкам; таким образом может быть определено положение судна в эти периоды, отмеченные на карте глубин. Изменения уровня поверхности воды (например, из-за прилива) также непрерывно регистрируются по отношению к некоторому нулевому уровню, давая возможность приписывать каждое увеличение глубины к этой характеристике. Обработка данных заключается в откладывании измеренных углов на карте, чтобы определить точки вдоль маршрута, которые снимают с эхограмм и графиков записи приливов и, сделав поправку на прилив, получают распределение измеренных глубин вдоль маршрута по отношению к времени. Эта часть программы является очень трудоемкой и дорогостоящей, если все эти операции выполняются вручную.

Лаборатория « SOGREAH » использует компьютер; устройство, считывающее кривые, и автоматический прибор для обработки данных и получает набросок карты и окончательную карту глубин с минимумом ручного труда.

Посредством этого стоимость обработки данных уменьшилась наполовину, тогда как время, требующееся на эту операцию, уменьшилось до одной четверти. Ппрограммы обработки данных являются гибкими и используют практически все методы сбора информации, которые применяются в этой области.

Tout sondage bathymétrique classique comprend une phase "mesures sur le terrain" et une phase "dépouillement en bureau".

L'embarcation, équipée d'un écho-sondeur, suit des routes et enregistre en continu la profondeur sur une bande de sondage. A intervalles réguliers, des opérateurs mesurent, par rapport à des repères connus, des angles qui permettront de déterminer la position de l'embarcation à des instants qui sont repérés sur l'enregistrement de l'écho-sondeur. On enregistre également les variations du plan d'eau pendant la période des mesures afin d'en connaître à tout instant la cote par rapport à un zéro de référence et de pouvoir rapporter chaque sonde à ce zéro (correction de marée, par exemple).

La phase "dépouillement" consiste à reporter les angles mesurés de façon à définir les points repérés de la route suivie, à dépouiller les bandes d'enregistrement de l'écho-sondeur et du marégraphe et à répartir, après correction de marée, les profondeurs mesurées le long de la route suivie en utilisant les repères de temps. Cette phase est très longue et grève considérablement le coût d'une campagne de sondage lorsque toutes ces opérations sont faites à la main. La SOGREAH utilise les possibilités d'un ordinateur, d'un lecteur de courbes et d'une table traçante pour traiter les informations recueillies et obtenir la minute de sonde et la carte bathymétrique avec un minimum d'opérations manuelles. Les programmes de traitement ont été mis au point pour le dépouillement des campagnes de sondage classiques en tenant compte du matériel employé actuellement. Nous verrons qu'ils sont directement transposables et trouvent même leur véritable finalité lorsqu'on traitera les campagnes de sondage avec du matériel digitalisé et embarqué donnant simultanément, et avec un pas de temps court, les mesures permettant de reconstituer les trois composantes x, y, z à chaque instant.

Le dépouillement d'une campagne de sondage classique comporte une chaîne de traitements que nous allons analyser.

PROGRAMME "TOPOGRAPHIQUE"

Les relevés topographiques sont notés directement par les opérateurs sur les bordereaux préétablis qui sont ensuite perforés. Le programme "topographique" détermine à partir des angles ou visées, affectés d'un indice temps, les coordonnées des points de localisation de l'embarcation X2, Y2, T2. Il existe évidemment plusieurs sous-programmes de calcul adaptés à la méthode de repérage topographique utilisée.

Ces coordonnées sont traitées sur la table traçante connectée à l'ordinateur. Le tableau des valeurs X2, Y2 est injecté dans le programme SOFTRA 4, qui est l'ensemble des sous-programmes de tracé permettant de commander nos traceurs à commande numérique et qui reconstitue le plan des routes suivies par l'embarcation. Par ailleurs, nous effectuons un quadrillage du plan de sonde pour permettre un repérage et le tracé de la côte si cela s'avère utile. Le choix de l'échelle du dessin est bien entendu défini avant l'appel des programmes de tracé.

Cette sortie intermédiaire constitue une vérification utile avant le dépouillement du sondage proprement dit. Elle permet de détecter, le cas échéant, certaines erreurs de navigation ou certaines erreurs de mesures topographiques et, éventuellement, de les corriger ou d'éliminer les zones incriminées.

Le programme fournit enfin le paquet de cartes perforées, ou l'enregistrement sur bande magnétique, des valeurs X2, Y2, T2, qui seront utilisées dans le programme "sondage".

DEPOUILLEMENT DE L'ECHO-SONDEUR ET DU MAREGRAPHE

Echo-sondeur

Les enregistrements se présentent sous forme de bandes sur lesquelles figurent deux types de renseignements : (a) les tops de position avec leur situation dans le temps (points topo) ; (b) le relief du fond.

Ces deux types de renseignements sont relevés et mis sur cartes perforées ou sur bande magnétique à l'aide d'un lecteur numérique de point type Benson 620. L'opérateur tape d'abord une première carte ou un premier enregistrement compor-

tant les renseignements généraux relatifs au segment à dépouiller, tels que : date, numéro de segment, type de bande employée, graduation en "feet" ou en mètres (ceci pour les conversions ultérieures d'unité).

La bande étant positionnée sur le lecteur, l'opérateur enregistre un système d'axes avec pour origine sur l'axe des x le zéro sondeur et sur l'axe des y le premier top de temps. L'opérateur prend de plus la précaution d'enregistrer une grande valeur sur ox et une autre sur oy pour avoir un système d'axes propre au segment et éviter une erreur en rotation entre les axes du segment et ceux du lecteur de points. Les points intéressants sont ensuite enregistrés normalement suivant les temps croissants, c'est-à-dire suivant les valeurs croissantes de x dans le système d'axes du segment. Les points enregistrés sont :

Les points représentatifs du relief, c'est-à-dire ceux qui suffisent pour reconstituer, par interpolation linéaire ou par lissage, le relief du segment avec une tolérance d'erreur choisie.

Les points relatifs aux tops de position, c'est-à-dire correspondants aux points de localisation traités par le programme "topographique" et qui sont enregistrés avec un code spécial.

Pour chaque segment nous avons donc un enregistrement comprenant : (a) les renseignements généraux relatifs au segment ; (b) le système d'axes dans lesquel sont enregistrées les coordonnées des divers points ; (c) les coordonnées des points sélectionnés du segment Xs, Ys.

Marégraphe

Les enregistrements du marégraphe, qui se présentent également sous forme de courbes, sont mis sur cartes perforées de la même manière. On enregistre successivement la date, l'heure de début et l'heure de fin d'enregistrement et les coordonnées de la courbe de marée à intervalles égaux : Tm, Zm.

Si la marée est bien connue et que des tables existent, ce sont des tables qui sont perforées par la méthode classique pour être introduites en mémoire.

PROGRAMME "SONDAGE"

Ce programme a pour but de construire la minute de sonde, c'est-à-dire de calculer les coordonnées X, Y, Z de chaque point de sondage à partir des trois types d'enregistrement établis précédemment : (a) points de localisation de l'embarcation résultant du programme "topographique" : X2, Y2, T2 ; (b) coordonnées résultant du dépouillement de l'écho-sondeur Xs et Ys qu'il convient de transformer en Ts, Zs ; (c) coordonnées de la courbe de marée, Zm, Tm.

Pour chaque segment de sondage, traité indépendamment, la liaison entre la localisation du bateau et la profondeur mesurée est assurée par la concordance entre les points de localisation et leurs correspondants parmi les points de sondage. En effet, nous avons vu que, pendant le dépouillement des bandes, les points de sondage correspondant à un top de localisation étaient repérés par un code particulier.

La vitesse et la route suivie par l'embarcation étant variables, le seul critère de référence valable est le repère temps des points de localisation. On suppose toutefois que la vitesse de l'embarcation est constante entre deux points topographiques successifs, ce qui permet de localiser chaque point de sondage par interpolation entre les deux tops topographiques qui l'encadrent. Cette approximation est justifiée par le fait que les intervalles temps sont au plus de deux minutes.

L'enchaînement des opérations par le programme est le suivant.

Opérations sur les données de marée. Pour effectuer sur chaque point de sondage la correction de la marée, il est nécessaire de discrétiser la fonction marée $Zm = f(T)$ par un sous-programme qui effectue localement la représentation analytique de la loi de marée en assurant la continuité des courbures (lissage avec tolérance). Les valeurs T, Zm et les dérivées de la fonction, dZm/dt, sont stockées en mémoire pour utilisation ultérieure.

En effectuant cette opération en tête du programme, on peut, dans l'hypothèse d'une grosse exploitation, faire une fois pour toutes l'opération de lissage de la marée et stocker les résultats sur mémoire annexe (bande ou disque), d'où gain de temps.

Opérations sur les points de localisation. Le seul critère de référence valable étant le repère temps des points de localisation, pour être indépendant de la route suivie nous établissons à partir des données X2, Y2, T2 deux fonctions paramétriques : $X2 = f(T)$ et $Y2 = f(T)$.

Par la même procédure de lissage que précédemment, nous obtenons et nous stockons : T, X2 et $dX2/dt$, T, Y2 et $dY2/dt$, et cela pour chaque segment traité séparément, ce qui permettra par la suite, connaissant l'heure du sondage, d'en interpoler ses coordonnées.

Opérations sur les coordonnées issues du dépouillement de l'écho-sondeur. Quatre traitements sont à effectuer successivement :

1. Transformation des Ys en Zs, c'est-à-dire des ordonnées lues sur la bande en mètres, compte tenu des renseignements généraux relatifs au segment (échelle, rotation des axes).
2. Transformation des Xs en Ts, c'est-à-dire des abscisses en temps par interpolation de l'abscisse du point entre les deux tops topographiques qui l'encadrent, les points de sondage correspondant à un top de localisation étant repérés par un code particulier.
3. Correction de marée sur les Zs, grâce aux fonctions de marée lissées et stockées précédemment. Cela est fait par un sous-programme spécial qui, en fonction des valeurs discrètes T, Zm, dZm/dt et du paramètre Ts extrait la valeur Zm correspondante. En fait, on refait localement la représentation analytique de la fonction marée. Cela permet de retrouver la vraie valeur de la cote du relief $Z = Zs + Zm$.

   A noter qu'il est possible d'introduire à ce moment une correction complémentaire sur la vitesse de propagation du son, en fonction de la température ou de la salinité, si nécessaire.
4. Calcul des coordonnées géographiques des points de sondage. Par le même programme que ci-dessus, mais en utilisant successivement les deux fonctions lissées $X_2(T)$ et $Y_2(T)$ calculées et stockées au moment du traitement des points de localisation, et le paramètre Ts du point de sondage, on calcule les coordonnées planes X et Y de chaque point. A ce stade, nous avons donc la position géographique de chaque point de sondage et la cote du relief par rapport à la ligne zéro.

Elimination de points en fonction de certains critères. On peut, compte tenu de la densité du relevé et pour ne pas perturber la clarté de la minute de sonde, être amené à supprimer quelques points avant le tracé. Si ce sous-programme est appelé, ces points sont ou ne sont pas éliminés en fonction de deux critères principaux combinés : dX une distance horizontale minimale et dZ une cote minimale. Exemple : soit

dD la distance horizontale entre deux points et dC l'écart de leurs cotes ; si simultanément $dD < dX$ et $dC < dZ$ le point ne sera pas représenté sur la minute de sonde et ne sera pas conservé pour la suite des opérations. Il y a d'autres variantes possibles à ce sous-programme d'élimination : ainsi éliminer tout point dont la cote ne s'écarte pas de dC par rapport à la ligne droite réunissant le précédent au suivant.

Dans le cas où les bandes de sonde sont dépouillées au lecteur de point comme indiqué ci-dessus, une sélection a déjà été opérée lors du traitement des bandes de l'écho-sondeur et le sous-programme de sélection est peu opérant. Par contre, lorsque l'écho-sondeur sera digitalisé et traité automatiquement, il fournira, avec un pas de temps petit, une très forte densité de points dont beaucoup seront superflus. Le rôle du programme d'élimination sera alors important pour ne conserver que les points représentatifs du relief et ainsi alléger notablement les opérations suivantes sans rien leur faire perdre de leur précision.

Tracé de la minute de sonde. Les tableaux des valeurs X, Y, Z des points de sondage retenus sont injectés dans le programme SOFTRA 4 du traceur numérique déjà utilisé pour reconstituer le tracé des routes suivies. Nous faisons appel essentiellement à des tracés de symbole et de report de valeurs prélevées en mémoire. Nous faisons écrire par nos traceurs la valeur de la cote, la position géographique du point étant matérialisée par l'emplacement de la virgule dans la valeur de la cote. Le fond de plan (quadrillage, échelle, repérage, tracé de la cote) est le même que dans le plan des routes suivies exécuté par le programme "topographique".

Enfin le tableau des valeurs X, Y, Z est stocké sur bande ou sur disque en vue de leur traitement par les programmes d'exploitation, d'après des besoins de l'étude.

TRACE DE LA CARTE BATHYMETRIQUE ; PROGRAMME ISOTRA

L'aboutissement de cette chaîne de dépouillement se concrétise par le programme ISOTRA. Ce programme est un produit spécifique de notre société, à caractère très général. Il permet de visualiser toute fonction de deux variables indépdantes, c'est-à-dire toute surface dans un espace à trois dimensions, par le tracé automatique des courbes isovaleurs ou des courbes de niveau.

Il suffit de lui fournir les coordonnées d'une série de points de mesure appartenant à la surface et représentatifs de celle-ci pour obtenir, sur la table à tracer automatique, la carte des courbes isobathes.

Il peut, sur demande, restituer sur mémoire annexe les informations permettant de reconstituer la surface calculée. Dans le cas que nous considérons, ce programme relit le fichier, stocké par le programme "sondage", des valeurs X, Y, Z et trace automatiquement la carte bathymétrique des courbes de niveau.

EXTENSION POSSIBLE DE LA CHAINE DE CALCUL

Une extension de la chaîne de calcul est prévue pour les études qui ne se satisferaient pas complètement de la minute de sonde et de la carte bathymétrique, mais nécessiteraient des études complémentaires réalisables à partir du fichier constitué par le programme "sondage".

Tracé de profils en long du fond sur un trajet déterminé par le programme LONTRA pour la pose d'une conduite ou l'implantation d'une digue, par exemple.

Calcul de cubature par le programme COMISO pour une étude de dragage par exemple, où le programme calcule le volume du dragage en fonction du tracé et des dimensions du chenal à créer.

ADAPTATION DE LA CHAINE DES PROGRAMMES DE DEPOUILLEMENT AU MATERIEL DIGITALISE DES MESURES

Les programmes de tracé automatique de la carte bathymétrique, de profils en long de relief et de calcul de cubature sont des programmes d'exploitation d'un intérêt certain et utilisables quelle que soit la méthode de sondage et de dépouillement utilisée. Mais il est clair que la pièce essentielle de la chaîne de dépouillement automatique est le programme "sondage", qui leur fournit la matière première à partir des relevés de terrain.

Nous avons vu la structure du programme "sondage" adaptée à une campagne de mesure traditionnelle exécutée avec le matériel dont nous disposons actuellement. Mais ce programme a été mis au point dans la perspective du matériel de mesure digitalisé que nous allons acquérir et dont l'usage va se répandre. Il est donc directement adaptable et trouvera sa véritable finalité dans ces nouvelles conditions.

Lorsque les données de mesure seront directement enregistrées par du matériel digitalisé embarqué, nous pourrons avoir sur un même enregistrement, et avec un pas de temps très court, les renseignements relatifs à la position géographique et à la profondeur mesurée. Le programme "topographique" sera adapté aux nouvelles méthodes de repérage (par fréquence d'ondes radio, par exemple) et transmettra directement le tableau des valeurs X, Y, Z*s*. La phase intermédiaire du dépouillement de l'écho-sondeur grâce au lecteur de courbes sera supprimée, ainsi que le calcul de concordance entre les mesures de localisation et les mesures de sondage. Les enregistrements faits à bord seront directement transmis à un programme regroupant les traitements "topographique" et "sondage". Ce dernier, notablement allégé, aura pour rôle essentiel, en plus de la correction de marée, de sélectionner parmi les nombreux points de sondage mesurés et dépouillés ceux qui sont véritablement représentatifs du relief, de façon à ne pas surcharger sans aucun gain de précision la minute de sonde et de façon à fournir aux programmes d'exploitation (ISOTRA, LONTRA et COMISO) le volume de données minimal et suffisant pour rendre économique leur exploitation.

C'est pourquoi les sous-programmes de sélection ont déjà été introduits dans le programme "sondage", bien que dans l'état actuel de notre matériel la sélection soit déjà en partie assurée par le dépouillement des bandes de l'écho-sondeur. Lorsque cet intermédiaire sera supprimé le programme prévu pourra assumer entièrement cette sélection.

Le dépouillement automatique sur ordinateur des campagnes de sondage réduit notablement le coût de la phase de dépouillement et les délais de fourniture des résultats. Par ailleurs, la limitation au minimum des interventions manuelles réduit considérablement les risques d'erreur ou de corrections abusives et non justifiées. L'élimination de ces risques et la rentabilité de la chaîne de calcul déjà reconnues pour l'exploitation d'une campagne faite avec du matériel classique sera encore mieux assurée lorsque la mesure elle-même sera automatique.

Developments in the processing of hydrological data in Australia

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Abstract. Over the last 4-5 years several water authorities in Australia have, in general, moved to the use of digital computers in processing hydrologic data. The use of chart conversion equipment for processing graphically recorded data is now almost standard and a start has been made in the use of recorders which employ computer-compatible media to record the data.

The authorities have used computers for rating current meters, calculating river discharges from current meter data, and computing daily and peak discharges, which are usually also stored on magnetic tape for subsequent analysis. In order to facilitate transfer of data between authorities with a wide range of computers and internal processing systems, standard interchange formats have been developed which can handle virtually any time-varying data, and comments, and permit the "tagging" of data to indicate quality attributes.

PROGRES DANS LE TRAITEMENT DES DONNEES HYDROLOGIQUES EN AUSTRALIE

Résumé. Durant les quatre ou cinq dernières années, les services chargés des jaugeages de courant se sont en général orientés vers l'usage d'ordinateurs digitaux pour traiter les données hydrologiques. L'utilisation d'un équipement de conversion pour l'analyse des enregistrements graphiques est maintenant presque complètement adoptée et l'on a commencé à utiliser des enregistreurs qui emploient du matériel compatible avec l'ordinateur.

Les autorités ont utilisé des ordinateurs pour calculer les vitesses de courant, les débits des rivières d'après l'information reçue des moulinets ainsi que le débit journalier et le débit maximal qui sont habituellement enregistrés sur ruban magnétique pour analyse ultérieure. Pour faciliter l'échange d'informations entre services utilisant une grande variété d'ordinateurs et de systèmes d'analyses internes, des formats normalisés facilitant les échanges ont été développés pour traiter virtuellement n'importe quelles données variables en fonction du temps et les informations complémentaires, et permettre l'étiquetage des informations en indiquant leurs qualités.

PROGRESOS EN EL PROCESO DE DATOS HIDROLÓGICOS EN AUSTRALIA

Resumen. A lo largo de los últimos cuatro o cinco años, las diversas autoridades del agua en Australia, en general, han tendido a utilizar ordenadores digitales en el proceso de datos hidrológicos. El uso de equipos de conversión de gráficos para someter a proceso los datos registrados gráficamente es ahora casi normal y se han empezado por usar registradores que emplean métodos de ordenador para registrar los datos.

Se tilizan ordenadores para el calibrado de medidores de corriente, calculando los caudales fluviales a partir de los medidores de corriente, y calculando los caudales diario e instantáneo que generalmente se almacenan también en cinta magnética para análisis posteriores. A fin de facilitar el paso de datos con una amplia gama de ordenadores y sistemas de proceso internos, se han puesto a punto formatos de intercambio normalizados que pueden servir virtualmente para datos variables con el tiempo que permiten el "marcado" de datos para indicar atributos cualitativos.

УСОВЕРШЕНСТВОВАНИЕ ОБРАБОТКИ ГИДРОЛОГИКЕСКИХ ДАННЫХ В АВСТРАЛИИ

Резюме. В течение последних 4-5 лет ряд водных управлений в Австралии в основном перешли на использование цифровых компьютеров при обработке

гидрологических данных. Ииспользование оборудования для трансформации карт в целях обработки графически записанных данных в настоящее время является почти стандартным, и началось использование регистраторов, в которых применяются средства, схожие с компьютером для регистрации данных.

Управления используют компьютеры для оценки гидрометрических вертушек, для вычисления расходов воды по данным вертушечных измерений, для подсчета суточного и пикового расходов, которые обычно записываются на магнитную ленту для последующего анализа. Для облегчения передачи данных между управлениями с широким диапазоном компютеров и внутренних систем обработки были разработаны стандартные обменные формы, которые могли включать в себе фактически любые данные, изменяющиеся во времени, и комментарии, и давали возможность « метить » данные, чтобы показать качественные характеристики.

INTRODUCTION

Within Australia responsibility for the collection and processing of rainfall data rests with the Commonwealth Bureau of Meteorology. Responsibility for surface and groundwater measurements is often shared between a number of independent agencies of each of the six States. The Commonwealth Bureau has responsibility for the Northern Territory and the Australian Capital Territory and is directly involved in the Snowy Mountains Scheme, and, in partnership with the three States concerned, in the River Murray Commission.

The establishment of the Australian Water Resources Council (AWRC) in 1962 led to closer and continuing co-operation between the Commonwealth Bureau and the States on these and related matters. The Council publishes information on stream gauging networks, and has drawn attention to deficiencies both in the total number of sites gauged and in their distribution amongst the different drainage basins (AWRC, 1963, 1967). Arising from AWRC recommendations for remedying this situation, Commonwealth Bureau and State Governments are currently engaged on an accelerated programme of installation of hydrologic stations. Over the last four to five years there has been a marked shift towards the use of computers for data processing and towards the use of computer-compatible media such as magnetic tape or punched cards for the storage of the data.

The AWRC has also organized a number of co-operative projects, important examples being the Representative Basins Project (AWRC, 1970) and a field study of evaporation from storages situated in different climatic regions of Australia. For projects of this nature vast amounts of data are being collected and distributed to various project leaders.

Against this background the AWRC arranged for the development and production of "Standards for the Interchange of Data" which are described in detail later. Current developments in the processing of hydrologic data in Australia are first reviewed in outline.

AUTOMATIC PROCESSING OF HYDROLOGIC DATA

General review

With the rapid expansion of measuring networks in recent years and the consequent need for incorporating automatic procedures, many authorities in Australia have

turned to digital recording equipment and enlisted the aid of a digital computer for centralized processing of the data. The general move towards use of digital methods has not been without its problems.

Over the past five years or so, most of the major stream-gauging authorities in Australia have been engaged in independent studies each aimed at evolving an over-all computer-based system for automatic processing of its stream-flow records and the conversion and integration of existing manuscript records into the system. A "systems analysis" approach has been adopted in most cases and much streamlining of past procedures has resulted. Not the least important aspect of these studies concerned problems of an administrative nature involving re-orientation of tasks and persuasion and re-training, at all levels, of staff steeped in the old methods.

In general, the systems evolved have had to provide alternative paths for processing data from a number of different sources, as follows:

1. Data such as daily-read staff gauge readings collected in manuscript form and usually transferred to punched cards for computer processing.
2. Chart records processed using analogue-to-digital conversion methods.
3. Output from digital recorders, recorded directly in the field on computer-compatible paper tape.

The authorities differ in the amount of processing carried out in a routine way. Some, for example, hold data on mean monthly flows in storage on magnetic tape and use this as input for tabulations and other analyses. Others store only daily values and compute monthly means or perform other reductions on the data as and when they are required. Generally, the types of processing carried out by computer programming include the following:

1. Current meter rating and production of rating tables.
2. Filing of digital, graphical and gauge-read water level records.
3. Processing of water level files to produce: (a) daily, monthly and annual flows; (b) monthly and annual maximum and minimum rates of flow; (c) flow duration tables; (d) summaries of the foregoing for compilation of a monthly data publication.
4. Stage height and flow hydrograph tabulations or graphical plots as required.
5. Calculation of reservoir inflows.

This list is by no means exhaustive. According to individual requirements, many other types of data reduction have been carried out. Some authorities, for example, compute various statistical parameters as a matter of routine, for applications in the field of stochastic hydrology. Mention should also be made of rapid advances in computer applications in the field of meteorology.

The development of electronic data processing methods has afforded authorities a much greater versatility in data handling capabilities and improved facilities for quality control. It has also provided the potential for wider accessibility of basic data for analysis and other purposes. There has been an increasing demand for transfer of hydrologic data between authorities and to other organizations which have some involvement in water resources assessment or research, and a growing realization that these demands can only be met through the services of a digital computer and transfer on computer-compatible media.

Conversion and storage of graphical data

The great majority of automatic recorders for the measurement of stream level, evaporation, rainfall, etc. are still of the graphical type. For computer processing the information must first be converted from analogue to digital form. To perform this

function a semi-automatic conversion unit, with features particularly suited to hydrologic applications, was developed within the CSIRO (Goodspeed and Savage, 1968). This device is now in commercial production and is extensively used by water authorities (and others). It is capable of handling charts produced by a wide range of recorders, including circular charts.

Various water authorities have developed their own computer-based data processing systems to handle the output of the conversion unit, converting the data to magnetic tape storage, producing tabulations and analyses. The details of these systems depend on the nature of the computer facility used and on the particular needs of each authority. Most are based on a general-purpose system developed concurrently with the conversion unit for use in a hydrologic research environment (Goodspeed, 1967).

The advent of remote terminals with time-shared access to a large computer has greatly facilitated data editing operations and has also simplified the operations involved in data analysis (Goodspeed, 1968). However, such facilities are unfortunately not yet available to the majority of water authorities in Australia.

Digital recording

If data processing is to be handled by digital computer, then it is obviously desirable that the original record be obtained in digital form. For a number of reasons, however, the proportion of hydrologic recorders of this type in Australia is still very small.

Some authorities have used the Fisher and Porter instrument which records on non computer-compatible paper tape. This tape is converted by a special translating machine into punched cards for input to the computer.

At the request of the AWRC, a digital event recorder was developed by CSIRO (Goodspeed and Savage, 1969). This instrument was originally designed for recording rainfall, but has been subsequently developed to record water level (and other variables) concurrently. Recording is on five-level computer-compatible paper tape. Thus no conversion is required before input to the computer. For long-term operation in remote localities the recorder is powered by nickel-cadmium batteries which are trickle-charged by solar cells.

The record obtained can be easily converted by computer into whatever storage format is used by each authority.

INTERCHANGE OF HYDROLOGIC DATA

Development of facilities for transfer

The Technical Committees of the AWRC established two working groups to study in detail the problems involved in uniform handling, storage and interchange of hydrologic data. It became patently obvious to the working groups that most authorities have individual and diverse requirements related to their own mission objectives which could not be met by any over-all standards and a system aimed at convenient and reliable transfer of data was eventually put forward. The method of approach adopted by the working groups and a general outline of the recommendations put forward have been discussed by Penny and Chapman (1967).

While this work represented a valuable step forward, increasing evidence that the specifications for interchange proposed by the working groups did not fully meet the

requirements of surface water authorities led to further deliberations as a result of which a completely revised format scheme was subsequently proposed. This development, in fact, endorsed an observation made by the original working groups that, in view of the current rate of developments in the electronic data processing field, any scheme put forward would need to be subject to constant review and this comment still applies.

General details of scheme

The revised specification defines standards for a generalized scheme for interchange of data on computer-compatible media, including formats, media and codes and an initial specification of material which should be available for transfer, together with recommended units. It was devised principally for *transfer* of data and compatibility between different computer systems is, therefore, a major consideration. The scheme treats time and variable data as more or less identically structured "items" of information and incorporates a data-type-and-quality index for "tagging" each pre-scaled element of information. Comments pertaining to the quality of the data may also be incorporated. Virtually all types of time-variable hydrologic data can be handled and the scheme is particularly suited to strip-chart conversion and event-recording methods.

General remarks on format and implementation

The preferred medium for data transfer will generally be magnetic tape. In the interests of compatibility it was stipulated that all data for interchange should be presented in records of 80 binary-coded-decimal (BCD) characters and the decision as to how many of these records can be "blocked" between inter-block-gaps on magnetic tape (with consequent saving in space) would be subject to a bilateral agreement between the interchange authorities. While the format specifications are intended primarily for transfer on magnetic tape, the 80-character record structure is readily amenable to logically-identical transfer on any one of the common media. The choice for transfer will be determined by what the holding authority can prepare and the requesting authority can accept.

Identification of data files

Labelling. Apart from a magnetic tape "system label", which may be mandatory for a particular processing system, a "descriptive label" constitutes the first 80-character record of a transmitted data file. This label contains information identifying the interchange authorities, date of creation of the file and appropriate cross-referencing material.

Manifest. To effect transfer of data a standard request form will be submitted in duplicate on which the requesting authority will set out its requirements. The sending authority will also record details including a precise copy of the descriptive label. Details of "system labels", if mandatory, are also included.

Standards for media and codes

Media. In view of the present state of development of computer recording media and current efforts to standardize on codes, it was recognized that any recording standards set down might need to be reviewed at frequent intervals. Seven-track, 0.5 in.

(12.7 mm) magnetic tape with "IBM-compatible" recording mode was stipulated as the preferred standard for interchange. However, nine-track, 0.5 in. "IBM-compatible" magnetic tape was also regarded as acceptable, although in most cases this would involve an additional step of mode transcription through the services of a suitably equipped installation.

Standard 1.0 inch (25.4 mm) wide punched paper tape (Standards Association of Australia, 1969a) and standard 80-column, twelve-row "Hollerith" cards were also regarded as acceptable media for transfer.

Codes. The standard FORTRAN character set was adopted for data transfer with seven-bit character codes and their implementation on the various media as specified (Standards Association of Australia, 1969b and c). The Australian standards are essentially identical with those proposed by the International Organization for Standardization and it was considered that there were obvious advantages in adopting standards corresponding to those in international use.

A six-bit BCD interchange code, first established by IBM and still widely used, may be implemented for interchange in the interim period prior to general adoption of the standard code.

Where an authority uses a different coding scheme to those specified, there should be little difficulty in carrying out appropiate conversion for interchange.

Format scheme. As has been indicated, the basic unit in the format structure recommended for transfer is the 80-character logical record. One or a number of such records are "blocked" into a single physical record or "block" on magnetic tape.

Within a complete data file, five classes of record are specified: label record, station header record, data header record, comment records, and data records.

The total data file transmitted may consist of one or a number of data sub-files, each commencing with a station header followed by a data header record. The station header record contains information which will identify the gauging station and the data header record identifies the type of variable being transmitted and the period of record covered in that data sub-file.

The type of variable transmitted is specified by a six-digit coding scheme which is systematized to permit handling of all types of data transformations (such as instantaneous, mean, extreme or total values) and is sufficiently broad to allow for all foreseable requirements. Space-variable as well as time-variable data are catered for in the scheme.

Time and date identification is specified by a simple eleven-digit code which permits resolution to the nearest minute.

Data records

Each data record consists of seven ten-character "items" of information and a ten-character sequence number field.

A data item specifies the value of either the dependent variable being transmitted or the independent variable (time or space). The first character of an item field is an index consisting of one of the digits 0 to 9, coded to indicate data type and quality attributes. The ramaining nine characters define the numeric value of that "item" (either time/space increment or variable element).

An index code in the range 0 to 3 designates variable data, zero indicating best accuracy possible in terms of the station instrumentation and the remaining values

providing data flags by which that data item may be referenced in earlier 80-character comment records in the data sub-file.

An index code in the range 5 to 9 designates independent variable data (time or space). A time entry refers to the *time interval* between the variable data entry immediately prior to this time entry and that immediately following, or the interval from the start time specified in the data header record, if it is the first item of a data sub-file. Specific values of the index code indicate the quality of definition of the instantaneous variation of the variable in terms of the sampling interval. A code value of 9, for example, indicates a period of missing record, i.e. it indicates that the element being transmitted varied in a significant manner, but no detail is available due to instrument failure or some other reason.

The index code 4 indicates the end of a data sub-file if the remaining entry in the item field is zero or the end of a total data file if the remaining entry is non-zero.

Data sequence

The data in a data sub-file will consist of a series of time (or space) increments and variable data with the last ten-character field in each group of eight (i.e. each record) containing a sub-file/record sequence number. Values of the variable may be transmitted at regular time intervals, in which case only the one time entry is required and it is assumed that all succeeding variable data entries occur at this frequency. However, in many instances, much space will be saved if variable time increment entries are recorded and data values are only logged when there is a significant change in the value of the variable. The sampling interval may be changed as frequently as desired within a data sub-file.

Operation and maintenance of the scheme

The interchange scheme commits each participating authority to the development of simple servicing programmes for extraction of data from its internal storage system and for receipt of data.

It also entails a central responsibility for a number of tasks necessary to ensure successful operation of the scheme.

The AWRC secretariat has accepted the responsibility of providing liaison for interchange where necessary, including a transcription service in cases where the interchange authorities do not have common media.

The secretariat has also undertaken to compile and maintain a reference catalogue, providing a list of media on which individual authorities can provide and accept data, including magnetic tape specifications, and a list of code assignments for hydrologic variables available for transfer.

At the present time the scheme is being implemented successfully in a major research project sponsored by the AWRC on the hydrology of small rural catchments. In this project one authority has been gathering and co-ordinating data from selected catchments distributed over the continent and is disseminating this data in computer-compatible form to a number of project leaders for analysis.

FUTURE DEVELOPMENT

Recent advances in techniques for processing of hydrological data in Australia have received much of their impetus from developments such as those described in this paper. The principle behind the semi-automatic conversion unit is now well esta-

blished and is likely to be retained for some years as the basis for processing graphical streamflow data. It is possible that long-term digital recording may eventually replace chart conversion in future moves towards more complete automation, but this requires further development to ensure adequate methods of quality control. Such prospects are attractive under Australian conditions of remoteness and inaccessibility, but would need to be justified on the grounds of clear and proven advantages. At this early stage in its implentation it seems reasonable to say that, together with the other developments, the proposed scheme for interchange of hydrologic data will go a long way towards satisfying the growing demand for quick and effective processing and transfer of hydrologic data in Australia. Further experience and future developments will almost certainly indicate areas where reassessment of techniques and methodology will become necessary to satisfy the demands of modern progress.

BIBLIOGRAPHY / BIBLIOGRAPHIE

Australian Water Resources Council, 1963. Review of Australia's water Resources.

—. 1967. Stream-gauging Information, 1965 (and subsequent annual supplements).

—. 1970. The Representative Basin Concept in Australia (a progress report).

Goodspeed, M. J. 1967. A conversion, storage and processing system for graphically-recorded data. In: *Collection and Processing of Field Data,* p. 567. E. F. Bradley and O. T. Denmead, eds., New York, Interscience.

—. 1968. A data collection and storage system using a digital event recorder. In: *The use of analog and digital computers in hydrology,* p. 473-484, Paris IASH – Unesco.

—. Savage, J. V. 1968. An analogue-to-digital converter for chart records. *J. Sci. Instrum.*, (Series 2), vol. 1, p. 451-455.

—. 1969. A multi-channel digital event recorder for field applications. *J. Sci. Instrum.*, (Series 2), vol. 2, p. 178-182.

Penny, J. P.; Chapman, T. G. 1967. Transfer of Hydrologic data on computer media between independent authorities. International Conference on Water for Peace, Washington.

Standards Association of Australia, 1969 (a). Dimensions for punched paper tape for Data interchange. Document 1442.

—. 1969 (b). Implementation of the 7-Bit coded character sets for information Processing Interchange. Documents 1437-1440.

—. 1969 (c). 7-bit coded character set for Information Processing Interchange. (AS XI – 1969).

Methods and means for preparing hydrological observation results for processing on computers

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Abstract. Important hydrological problems have arisen from the fact that new ways for the rational use, exploitation and protection of water resources and land reclamation have been sought. To solve these problems, the amount of hydrological information available must be increased, its quality and processing improved, and the transmission of observational data speeded up. Rapid and high-quality processing of hydrological information is possible only by means of modern computing facilities and, therefore, observational data must be programmed by means of punched cards, punched tapes or special blanks for use by computers. Because of some technical characteristics, punched tape has essential advantages over other technical media, as well as being more convenient for storage and transmission. Data can be recorded onto punched tape from hydrological logbooks either by hand or automatically. By using punched tape it is possible to construct a flexible lay-out including all available data. This is very difficult to attain when using punched cards. However, neither punched cards nor punched tapes completely meet the necessary requirements for long-term storage of information. In this case microfilm is used for the creation of an easily-available fund of hydrological data.

METHODES ET MOYENS DE PREPARATION DES RESULTATS DES OBSERVATIONS HYDROLOGIQUES POUR TRAITEMENT PAR ORDINATEURS

Résumé. L'utilisation et l'exploitation rationnelles des ressources en eau, leur protection ainsi que la mise en valeur économique de nouveaux territoires représentent des tâches importantes pour les hydrologues. La résolution de ces problèmes exige un traitement de l'information hydrologique par des moyens augmentant son volume, améliorant sa qualité et accélérant la vitesse de transmission des résultats d'observation et le perfectionnement des méthodes de traitement. On ne peut obtenir un traitement rapide et de haute qualité des données hydrologiques que par la mise en œuvre de calculateurs modernes, tous les résultats d'observations étant enregistrés sur le porteur technique. Comme porteur, on peut utiliser des cartes perforées ordinaires ou des bandes perforées et des papiers spéciaux. La bande perforée présente des avantages par rapport aux autres porteurs en raison de certaines caractéristiques techniques. L'information peut être enregistrée sur bande perforée de façon automatique ou manuelle à partir des registres. Il n'est pas nécessaire de mettre en œuvre la technique de gros gabarit, l'archivage et la transmission de l'information étant plus commodes. L'utilisation de la bande perforée donne la possibilité d'avoir un enregistrement souple qui contiendra le volume total d'observations, ce qui ne peut être obtenu au moyen des cartes perforées. Mais ni les cartes perforées ni les bandes perforées ne satisfont entièrement aux besoins de l'archivage de longue durée. A cette fin, on utilise le micro-film, qui permet de créer un fonds de données hydrologiques facilement utilisable.

MÉTODOS Y MEDIOS PARA LA PREPARACIÓN DE RESULTADOS DE OBSERVACIONES HIDROLÓGICAS PARA SU PROCESO EN ORDENADORES

Resumen. La exploración de los caminos para la utilización racional de los recursos de agua, su explotación y protección razonable, y la utilización económica de nuevos territorios plantean importantes problemas en hidrología. A fin de resolver estos problemas, es necesario un ulterior desarrollo de la información hidrológica, por medio del aumento de su volumen total, mejora de su composición cualitativa, aceleración de la transmisión de resultados y mejora de sus métodos de proceso. La puesta al día y el proceso de alta calidad en la información hidrológica es posible sólo por medio de modernos ordenadores electrónicos, y por tanto los resultados de observación

se pueden registrar más técnicamente. Para ello se utilizan tarjetas perforadas comunes, cintas perforadas o formatos especiales. En función de determinadas características técnicas, la cinta perforada posee ventajas esenciales con respecto a otros medios técnicos. El registro de datos sobre ella se podría hacer automáticamente y a mano, a partir de los libros de registro. Para estos fines no se necesita equipo pesado; la cinta perforada es más conveniente para el almacenamiento y transmisión. El empleo de cinta perforada permite la posibilidad de tener un mecanismo flexible que incluya el volumen completo de datos, lo cual no se logra con la utilización de tarjetas perforadas. Pero ni las tarjetas perforadas, ni tampoco las cintas satisfacen por completo las exigencias de la recopilación de información a largo plazo. Por tanto, se utiliza microfilm para crear un fondo automático de datos hidrológicos.

МЕТОДЫ И СРЕДСТВА ПОДГОТОВКИ РЕЗУЛЬТАТОВ ГИДРОЛОГИЧЕСКИХ НАБЛЮДЕНИЙ К ОБРАБОТКЕ НА ВЫЧИСЛИТЕЛЬНЫХ МАШИНАХ

Резюме. Разработка путей рационального использования водных ресурсов, разумной их эксплуатации и охраны, хозяйтсвенное освоение новых территорий ставят перед гидрологией новые важные задачи. Для решения этих задач требуется дальнейшее развитие гидрологической информации путем увеличения ее объемов, улучшения качественного состава, ускорения процесса передачи результатов наблюдений, усовершенствования методов их обработки. Своевременная и высококачественная обработка гидрологической информации возможна лишь при использовании современных вычислительных машин, в связи с чем результаты наблюдений должны быть занесены на технический носитель. В качестве таких носителей могут использоваться обычные перфокарты, перфоленты или специальные бланки. По некоторым техническим характеристикам перфолента имеет существенные преимущества перед другими носителями. Занесение данных на нее может осуществляться как автоматически, так и вручную с полевых книжек. Для этого не требуется крупногабаритной техники, ленту удобнее хранить и пересылать. Использование перфоленты обеспечивает возможность построения гибкого макета записи с включением в него полного объема наблюдений, чего не удается достичь при использовании перфокарт. Однако ни перфокарты, ни перфоленты не удовлетворяют полностью потребностям длительного хранения информации. По этой причине для создания фонда памяти гидрологических данных используется микрофильм.

Rational exploitation of water resources and economic cultivation of new territories necessitate improvements in both the quantity and quality of hydrological information; the process of transferring observations, the improvement of techniques and data processing must all be accelerated, and this can only be achieved by computers. In the Soviet Union, the volume of hydrological information is tremendous and special attention is paid to this problem. Automatic devices for carrying out observations are being devised, new communication channels for transferring operational data are being built and data processing centres equipped with modern computing machinery, including means of storage and automatic retrieval, are being created. The technical equipment of the data collecting system allows, in addition to the information from ground stations, the processing of satellite data which considerably supplement our knowledge of the hydrological régime. However, as well as the automatic methods of hydrological observations, common devices will probably be used for a long period of time. While information from automatic devices can be immediately recorded in a form suitable for subsequent computer processing, data received during non-automated observations need preliminary recording on punched cards, punched tapes and special blanks for coding information.

Considerable investigation and experimentation into the mechanized processing of hydrometeorological data have recently been carried out in the Soviet Union. In particular, some methods have been developed for placing the results of hydrological observations on technical media; the observations from non-automatic posts were first recorded in common logs and registers, then data were placed on a technical medium. It is obvious that in this case two variations for organizing work are possible: (a) centralized preparation of data for technical media; and (b) entering data on the technical media directly at the point of observation. In the first case, punched cards and punched paper tapes, on which the data are presented in the form of punch combinations, are used.

The 80-column punched card with data in decimal code is considered to be the most widely used, not only in the Soviet Union, but also in other countries. To make the process of punching easier, a document itself, as a rule, must be adapted in a special way. Usually one can indicate in the document a punched card column number where this or that figure should be placed. The entering of information on punched cards is performed semi-automatically with the help of perforators. However, a punched card does not always meet all the requirements of mechanized processing. First of all it only allows a rigid lay-out of data, with a quite definite upper limit to the volume. Exceeding this volume, in the case of data for one observation term or results of a complete measurement complex, considerably complicates data processing, especially in the punching stage.

In some cases the placing of information on a common punched paper tape turns out to be more suitable. According to technical characteristics (record density, input speed), a punched tape has considerable advantages as compared with a punched card. Data may be recorded on it either automatically or by hand from field log-books. A tape is convenient for storage and transport. The use of a punched tape provides the possibility of constructing a flexible record lay-out to hold a complete observation volume which cannot be contained on punched cards. They are prepared on simple and cheap punching apparatus. The continuity of a reel ensures that no single data element can be lost or replaced.

The use of a punched card or a punched tape as a technical medium during their preparation at a processing centre requires the expenditure of additional time and money. For reducing these expenses in the U.S.S.R., as well as in a number of other countries, media are being sought which can be prepared at the observation points without using expensive technical means. A punched card for graphite marks can be considered an example of such media. It consists of standard 60-column punched card on which oval marks are printed. A record on such a punched card is made by shading the oval marks with a soft pencil or electroconductive ink. The punched cards are then transferred to the processing centre where the transformation of recorded information is performed by means of a reading perforator at a speed of 6,000-7,000 punched cards per hour.

The designing of devices for immediate reading of information from primary documents, with consequent automatic placing on technical media or immediate input into a computer, has been recently carried out in many countries. However, the creation of reading devices capable of processing a variety of existing primary papers, which differ both in size and form, is technically very complicated and economically unprofitable. Limited progress is reached with the use of typified blanks filled with specially coded or stylized prints. Under field conditions the recording of the results of hydrological observations in such a way is very difficult and for this reason special blanks cannot be widely used.

In choosing a medium it is necessary to take into consideration the requirements at different stages of collecting, processing and storing of hydrological information. For example, although the media considered above are quite suitable for collecting and primary processing of observations, due to small record density, small wear-stability and low speed of data input into a computer, they turn out to be of little effectiveness for durable storage and the statistical processing of large volumes of data. It should be noted that, with increasing record density, the difficulties of making corrections in information increase. At the same time, the necessity for corrections during the manual work frequently occurs. Thus the process of recording data on a compact medium is often realized at a later stage. At first, data are placed on an intermediate medium which permits their preparation under diverse conditions, as well as allowing the possibility of checking information. As a rule, the primary results of observations are placed on such media. During the processing on computers, data undergo automatic quality control and are transferred to compact and durable medium for the purposes of storage.

Magnetic tapes have high record density and a high speed of input of data into computers. However, magnetic tapes have the substantial shortcoming that information on them can be easily lost and, with time, magnetic signals deteriorate because of pollution, temperature fluctuations and humidity changes. The period of reliable data storage on magnetic tapes does not exceed ten years and during this time it is necessary to take measures against the occasional erasing of information. Magnetic tapes can be used for collection and storage of processed results or regularly recieved observations before their being entered on a medium suitable for storage.

A microfilm of discrete information in a digital code is a reliable medium for an archival storage. It is compact and its speed of data input into a computer is comparable with that of input from a magnetic tape. Under corresponding conditions it provides information storage for from 100 to 150 years. However, the equipment used for the production of a microfilm and its reading is rather expensive.

Taking into account the parameters and the properties of the technical media described here and after carrying out experimental work, microfilm has been adopted as a medium of hydrological data for long-term storage in the U.S.S.R. Information is first sorted and stored on magnetic tapes which serve as temporary accumulators.

Punched cards and punched tapes are adopted as primary media of hydrological information. The results of hydrological observations of previous years, which are obtained from publications, are recorded again on punched cards. The results of observations after the introduction of the system of automated processing are placed on a punched paper tape, periodically processed on a computer and then recorded on magnetic tape. The errors are detected automatically by means of a special quality control system. Punching is performed directly from the observation registers. At the present time, the results of observations of water level, water temperature, ice phenomena, precipitation, discharge and suspended load, chemical composition of water and others are automatically processed. To make the work of punching more convenient, a special system of tagging has been produced, according to which every column of a register or a table has its own ordinal number corresponding to a tag of a memory cell. Such a method permits corrections to be inserted on a tape when errors appear during the process of manual punching.

Data stored on a tape, after correcting errors, are annually microfilmed. Thus, as output one can see a single row on microfilm for the whole period of observations. Figure 1 indicates clearly the use of different information media at different stages of reception, transfer and processing. The order in which hydrological information is

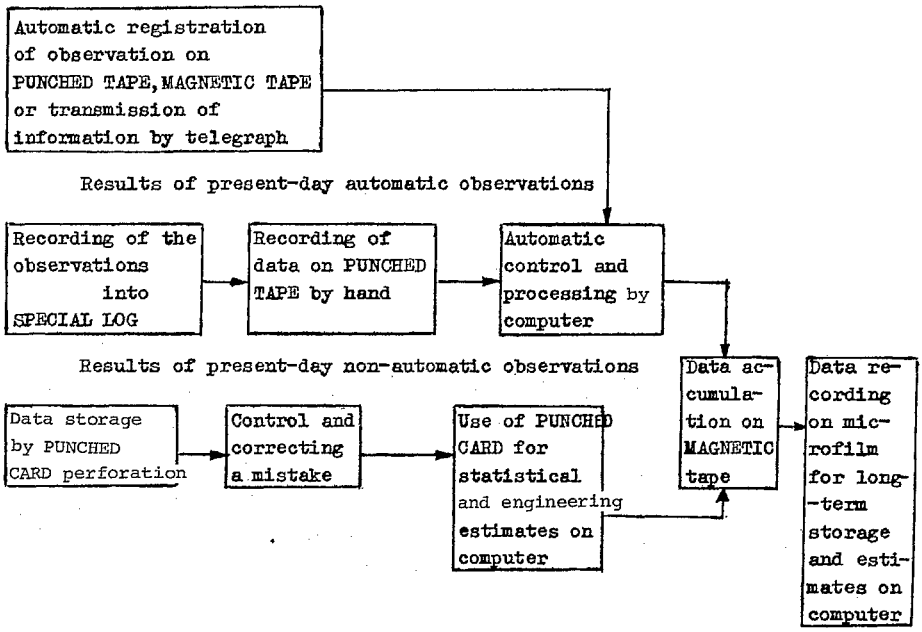


Figure 1 – *Scheme of data transformation and processing of hydrological information by computer*

Дата	Номера скоростных вертикалей, интервалов урезов и границ мерн. участка	Расст. от постоянного начала, м	Глубина, м		Угол относительно трассы	Высота подвеса, м
			1	2	глубина погруж. мерн. груза, м	глубина погруж. мерн. груза, м
1	2	3	4	5	6	7
2565)						
2573)						
2601)						
2607)						
2615)						
2623)						
2631)						

Figure 2 – *Fragment of a page of a log-book for the recording of discharge data and their perforation on punched tape*

A

Data cipher	Number of a river basin	Number of an observation point	Year	Number	Half year	Daily water discharge						Blank column
						I/VII	II/VIII	III/IX/	IV/X/	V/XI/	VI/XII/	
1-3	4,5	6-14	15,16	17,18	19	20-29	30-39	40-49	50-59	60-69	70-79	80

B

Data cipher	Number of a river basin	Number of an observation point	Year	Number	Daily water levels																		Blank columns
					1	Ice phenomena	II	Ice phenomena	III	Ice phenomena	IV	Ice phenomena	V	Ice phenomena	VI	Ice phenomena	VII	Ice phenomena	VIII	Ice phenomena	IX	Ice phenomena	X	Ice phenomena	XI	Ice phenomena	XII	Ice phenomena	
1-3	4,5	6-14	15,16	17,18	19-22	23	24-27	28	29-32	33	34-37	38	39-42	43	44-47	48	49-52	53	54-57	58	59-62	63	64-67	68	69-72	73	74-77	78	79, 80

Figure 3 – *Scheme for placing hydrological data on punched card*

placed on technical media as shown in Figure 3 shows the scheme for placing data on a punched card and Figure 2 a fragment of a logbook.

The system of automatic processing of the results of hydrological observations which has been developed in the U.S.S.R. using punched tapes, punched cards, magnetic tapes and microfilms, is effective compared with manual data processing. It allows a reduction in the time of processing observations together with an improvement in their quality.

Evaluation des fluctuations de vitesse, mesurées par micromoulinet et enregistrées sur bande d'oscilloscope

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Résumé. La communication présente le principe de construction et l'utilisation des abaques qui rendent possible l'évaluation graphique des bandes d'enregistrement obtenues au cours de l'étude des fluctuations de vitesse mesurées à l'aide d'un micromoulinet hydrométrique donnant des impulsions électriques.

L'utilisation de tels abaques rend possible la valorisation ultérieure d'enregistrements anciens de mesures effectuées sur le terrain ou autour des constructions hydrotechniques, en vue d'obtenir des renseignements précieux sur l'effet de la fluctuation macroturbulente de la vitesse. On peut ainsi obtenir de nouvelles données, même non prévues encore à l'époque des mesures.

EVALUATION OF OSCILLOGRAMS OF WATER VELOCITY FLUCTUATIONS MEASURED BY AN IMPULSE-GENERATING MICRO-CURRENT METER

Abstract. The principles of construction and utilization of tables which make it possible to evaluate graphically recordings obtained during studies of velocity fluctuations measured by a micro-current meter generating electrical impulses are described. The use of these tables enables the later evaluation of older registered data measured in hydrotechnical constructions with a view to obtaining precise indications of the macroturbulent fluctuations of velocities. Further new data can be obtained, even if their evaluation was not foreseen at the time of measurement.

EVALUACIÓN DE LAS FLUCTUACIONES DE VELOCIDAD MEDIDAS POR MICRO-MOLINETES Y OBTENIDAS POR OSCILOGRAMA

Resumen. En la medición de fluctuaciones de velocidad se utilizan medidores de tipo tornillo, de dimensiones e inercia reducidas.

El registro se obtiene con ayuda de un osciloscopio, prácticamente sin inercia, equipado con un foto-registrador. En la cinta de papel impresionable a la luz las marcas proporcionales a las revoluciones por segundo y las de tiempo se pueden determinar por medio de un nomograma realizado con estos fines.

Al deslizar el nomograma en la cinta de registro se pueden obtener directamente en un gráfico los valores instantáneos de la velocidad en función del tiempo.

ОЦЕНКА ФЛЮКТУАЦИИ СКОРОСТЕЙ, ИЗМЕРЕННЫХ МИКРОВЕРТУШКОЙ И ЗАРЕГИСТРИРОВАННЫХ ЛЕНТОЧНЫМ ОСЦИЛЛОГРАФОМ

Резюме. Использованы измерения скоростных флюктуаций, полученных вертушками лопастного типа малого размера и с малой инерцией. Регистрация осуществлялась с помощью практически безынерционного осциллографа, снабженного фотозаписывателем. На фоточувствительной бумажной ленте метки, пропорциональные числу оборотов в сек. и времени, могли быть определены с помощью специально сделанной для этого номограммы.

Скользящая номограмма записанных на ленте мгновенных значений скорости может быть немедленно вычерчена в фуикции от времени на чертежной бумаге, помещенной рядом с ней.

Les laboratoires d'hydraulique consacrent beaucoup d'efforts à l'étude de la turbulence. Ces études ont pris de l'essor depuis une dizaine d'années. Aujourd'hui, on dispose de connaissances approfondies sur la structure de la turbulence et son développement grâce aux études de caractère scientifique effectuées en laboratoire. Dans l'état actuel de nos connaissances, l'effet de la fluctuation turbulente ne peut être déterminé que par des mesures de vitesse ou de pression. L'utilisation du fil chaud et du film chaud a fait progresser les recherches fondamentales effectuées dans les conditions du laboratoire (eau pure, profondeurs réduites, visibilité parfaite, instrumentation adéquate, calculateurs électroniques, etc.).

En ce qui concerne l'étude sur modèles réduits ou sur le terrain des problèmes pratiques liés à la turbulence, la situation dans le domaine des mesures est beaucoup moins favorable. On utilise des micromoulinets de fabrications diverses pour la mesure des vitesses et des fluctuations turbulentes.

Parmi ces micromoulinets, quelques-uns sont pourvus d'un système électronique rendant possible non seulement la mesure et l'enregistrement des faibles vitesses et des fluctuations de vitesse, mais aussi le dépouillement rapide des données enregistrées. Ils sont assez fragiles, et ne sont utilisés que dans les conditions favorables que présente une étude en laboratoire.

Les micromoulinets OTT-Minor et Neyrpic sont suffisamment robustes et en même temps assez précis pour qu'on puisse les utiliser dans les conditions plus difficiles des études de laboratoire où l'on ne peut éliminer les difficultés que présentent les impuretés et les matériaux solides dans l'eau, les grandes dimensions des ouvrages ou du modèle de ceux-ci, les vitesses élevées, le manque de visibilité, etc.

Ces moulinets hydrométriques donnent des impulsions électriques qui peuvent être enregistrées sur bande de papier ; si l'on veut éviter le dépouillement des enregistrements, on utilise des compteurs électromécaniques pour totaliser les impulsions dans un intervalle de temps donné.

Si l'on veut connaître et étudier les fluctuations de la vitesse, l'enregistrement sur bande de papier constitue la seule solution. Dans ce qui suit nous présentons une méthode qui rend possible le dépouillement de tels enregistrements.

Au cours de l'étude sur modèle réduit d'un dissipateur d'énergie, les vitesses instantanées ont été mesurées dans différents points de plusieurs profils transversaux à l'aide d'un moulinet de petites dimensions du type OTT-Minor (hélice utilisée : n° 2-3). Ce moulinet fournit une impulsion à chaque tour de l'hélice.

L'enregistrement des impulsions est fait sur bande de papier photographique à l'aide d'un oscilloscope muni d'une photo-caméra. La figure 1 présente le schéma de principe de la chaîne d'enregistrement, tandis que la figure 2 donne la bande de papier sortie de la photo-caméra. Les lignes verticales représentent les temps, et les traits horizontaux de différentes longueurs correspondent aux impulsions données par le moulinet. Chaque trait plein (noir) représente un demi-tour d'hélice. La variation de leur longueur est fonction de la variation de vitesse du liquide. La solution adoptée rend possible un enregistrement pratiquement sans intertie, ce qui est très important pour les fluctuations rapides de la vitesse.

L'étude graphique des bandes a été résolue à l'aide d'un abaque spécialement construit et dessiné sur plaque transparente (celluloïd). La figure 3 montre un tel abaque, dont les caractéristiques sont les suivantes : moulinet OTT-Minor ; hélice n° 2-3 ; vitesse de déroulement de la bande de papier : 9,15 cm/s.

Le principe de construction d'un tel abaque est donné sur la figure 4. Le graphique *a* de cette figure représente la courbe de tarage du moulinet. Chaque valeur de n correspond à une valeur de la vitesse v.

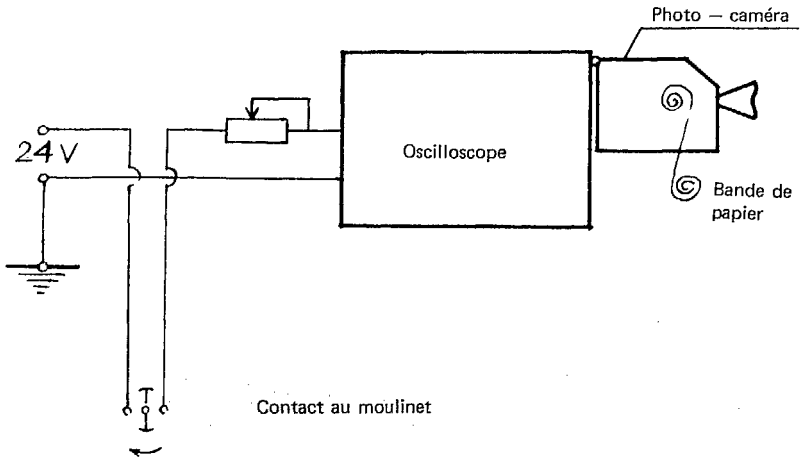


Figure 1 – Principe de la chaîne d'enregistrement des impulsions du moulinet à l'aide d'un oscilloscope muni d'une photo-caméra

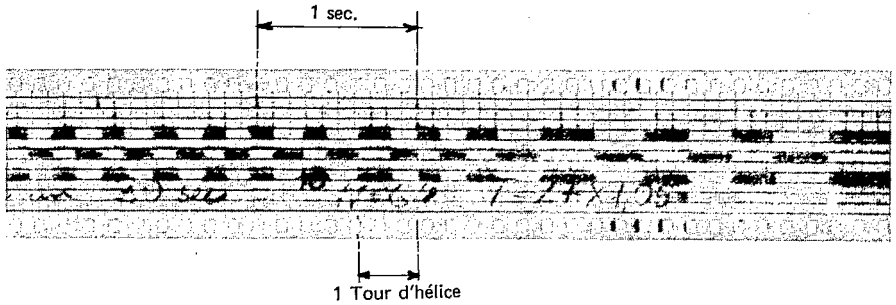


Figure 2 – Bande de papier sortie de la photo-caméra

La longueur L sur le graphique *b* – représentant la bande de papier – dépend de la vitesse de déroulement C du papier et donne en même temps l'échelle du temps. Cette vitesse sera choisie en fonction des vitesses de fluctuation et selon la précision voulue de l'évaluation graphique, parmi les vitesses de déroulement disponibles de la photo-caméra. Elle reste constante pendant la durée d'un enregistrement. Les longueurs l_1, l_2, et l_3 des traits noirs, représentant un demi-tour de l'hélice, dépendent de la vitesse de déroulement C du papier et du nombre de tours par seconde. Donc les vitesses v_1, v_2 et v_3 du graphique *a* sont transformées et visualisées sur le graphique *b* en longueurs l_1, l_2, *et* l_3.

Sur l'axe vertical du graphique *c* seront portées à une échelle appropriée et librement choisie les vitesses mesurées par le moulinet : v_1, v_2, v_3 . . . A la hauteur de chaque vitesse v_1, v_2, v_3 figure en abscisse la longueur respective : l_1, l_2, l_3. Les deux pointillés reliant les deux extrémités des l_1, l_2, l_3 . . . constituent l'une des paires de courbes de l'abaque d'évaluation. Cette paire de courbes se réfère à des signes qui correspondent à un demi-tour d'hélice.

De la même manière, on peut construire les paires de courbes valables pour 1, 2 et même pour 3 tours d'hélice, dont l'utilisation devient nécessaire pour l'évaluation des vitesses plus élevées.

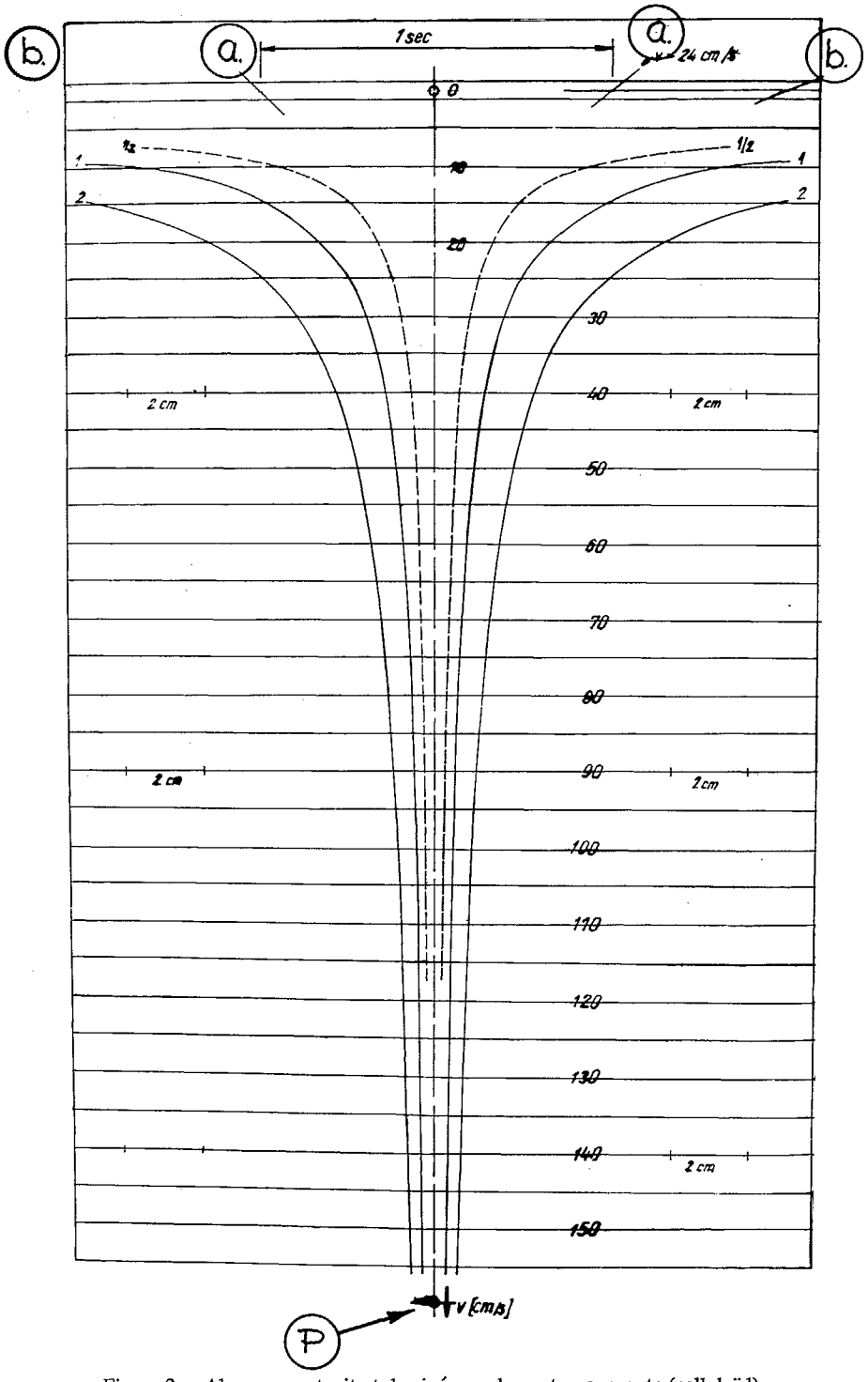


Figure 3 – Abaque construit et dessiné sur plaque transparente (celluloïd)

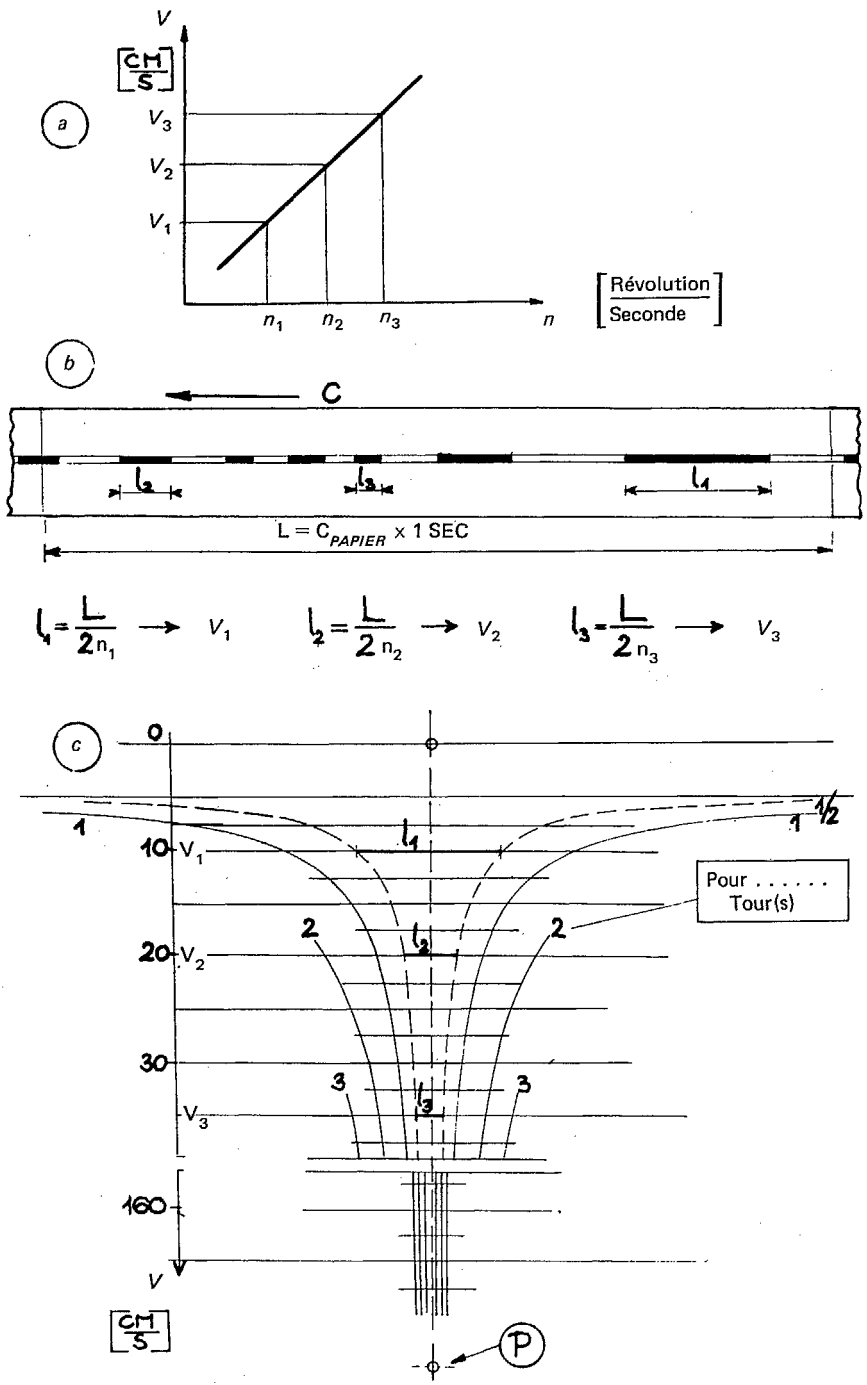


Figure 4 – Principe de construction d'un abaque sur plaque transparente (celluloïd).
a. Courbe de tarage du moulinet.
b. Bande de papier.
c. Vitesses mesurées par le moulinet.

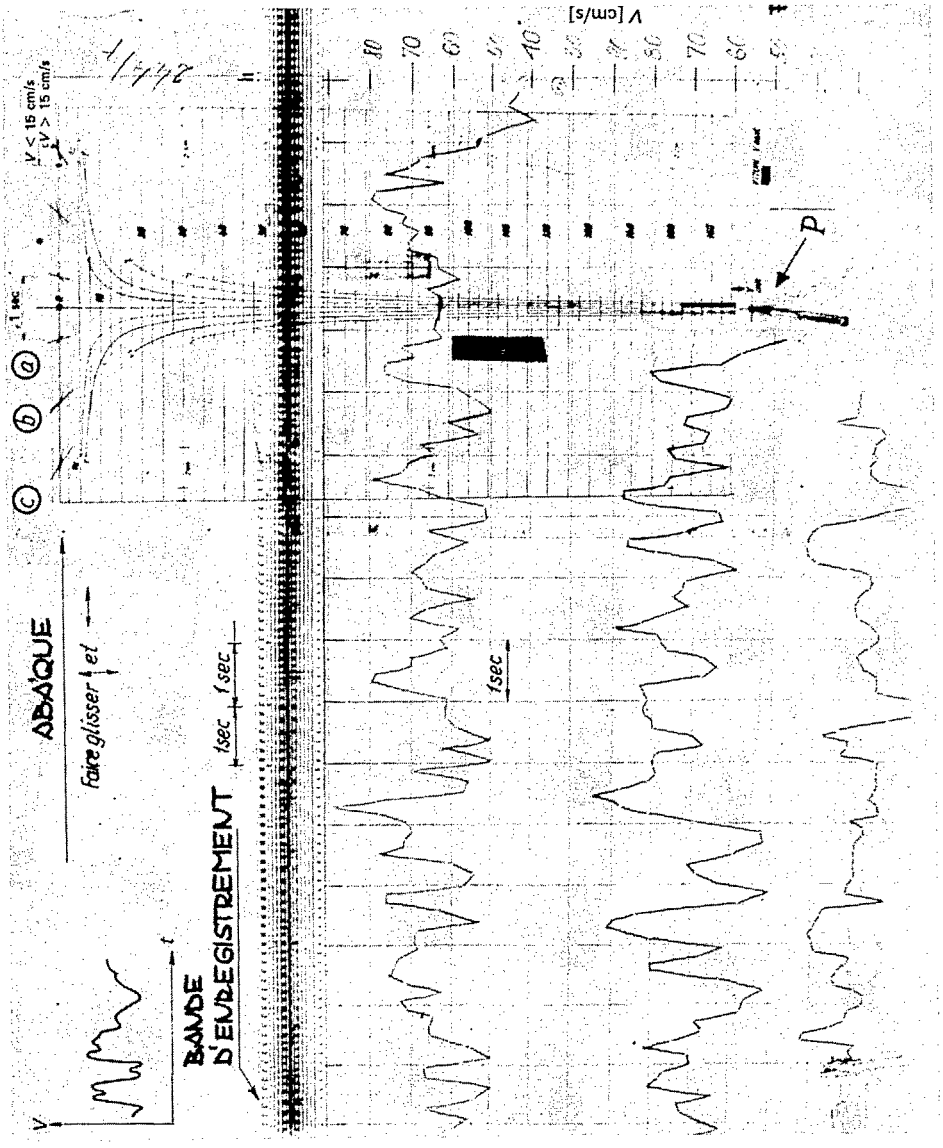


Figure 5 – Utilisation de l'abaque

A une distance librement choisie de l'axe $v = 0$ (cm/s), au point P se trouve un trou à travers lequel on marque, à l'aide d'un piquoir, le point évalué de la vitesse instantanée.

A chaque hélice et vitesse de déroulement du papier appartient un abaque particulier construit avec leurs paramètres respectifs.

La figure 5 présente l'utilisation de l'abaque.

En haut du papier à dessin sera placée la bande d'enregistrement fixée dans la position indiquée. Il est important que les lignes horizontales qui se trouvent sur la bande (voir fig. 2) soient très précisément des droites. On place l'abaque dessiné sur celluloïd sur la bande, de telle manière que les lignes horizontales sur le celluloïd soient parallèles aux lignes horizontales de la bande et, ensuite, on fait glisser le celluloïd dans le sens vertical jusqu'à la ligne $v = 0$, ou bien jusqu'à une autre ligne v, de façon que la vitesse minimale parmi les vitesses instantanées enregistrées coïncide avec l'axe horizontal des signes enregistrés.

Par une piqûre au point P, on marque la position de la ligne horizontale de la vitesse correspondante. On marque de la même manière d'autres horizontales pour d'autres valeurs de la vitesse et on les trace parallèlement à l'axe de la bande d'enregistrement placée dans le haut du papier à dessin.

L'échelle de temps est déterminée par la vitesse de déroulement C de la bande d'enregistrement. On trace donc des lignes perpendiculaires représentant des intervalles de 1 seconde marqués sur la bande. Ainsi le système de coordonnées est prêt et l'on peut procéder à l'évaluation point par point des valeurs instantanées de la vitesse fluctuante.

On procède de cette façon par demi-tour d'hélice si les fluctuations sont rapides, ou par un et parfois deux tours d'hélice s'il n'y a pas de fluctuation appréciable et si les vitesses enregistrées sont assez élevées.

On fait glisser le celluloïd de l'abaque dans le sens vertical et horizontal jusqu'à ce que les paires de lignes pour 1/2, 1 ou 2 tours coïncident avec les deux extrémités des signes correspondant à 1/2, 1 ou 2 tours d'hélice (voir fig. 2). La bonne position de l'abaque est atteinte si la coïncidence est parfaite et si les lignes horizontales et verticales dessinées sur celluloïd sont parallèles aux lignes respectives du système de coordonnées sur le papier à dessin.

Par une piqûre au point P de l'abaque, l'évaluation est terminée : la valeur instantanée de la vitesse est fixée en fonction du temps.

On passe au prochain signe de la bande et en espaçant les points selon la précision désirée, en piquant toujours en P.

En reliant les piqûres, on obtient le déroulement dans le temps des fluctuations de vitesses comme le montre la figure 5.

En haut de l'abaque (voir fig. 3 et 5), l'abscisse $v = 0$ est traversée par les paires de lignes *a, b,* et *c,* ce qui rend possible une correction dans la préparation du système de coordonnées dessiné sur le papier : le zéro de l'échelle pour les vitesses figurant sur l'ordonnée doit être déplacé vers le haut ou vers le bas jusqu'à ce que la paire de lignes *a* (ou *b* et *c*) coïncide (nt) avec la ligne des signes de la bande d'enregistrement, c'est-à-dire les verticales qui marquent respectivement le commencement et la fin des intervalles de temps de 1 (2 et 3) seconde (s).

Cette correction devient nécessaire si la vitesse de déroulement du papier diffère un peu de la vitesse prévue à la construction de l'abaque, ou bien si la bande d'enregistrement a subi des allongements longitudinaux pendant le développement photographique.

Les abaques présentés ont donné entière satisfaction pour les études effectuées sur modèles réduits de grandes dimensions.

Ils peuvent être utilisés même pour des études *in situ,* c'est-à-dire pour l'évaluation des fluctuations de vitesses mesurées dans la nature à l'aide des moulinets conventionnels dans les fleuves, rivières et autour des constructions hydrotechniques, etc. Leur utilisation rend possible la valorisation ultérieure des enregistrements anciens du point de vue des fluctuations de vitesse, c'est-à-dire qu'on peut les exploiter pour obtenir des données nouvelles précieuses, même lorsqu'elles n'étaient pas prévues à l'époque des mesures et de l'enregistrement.

Analogue to digital conversion and data acquisition from charts of water level and rainfall recorders and their evaluation by a computer

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Abstract. Manual and individual evaluation of water level recorders is not only a qualitative, but a quantitative, problem. Often there is not enough time or personnel for exact evaluation of all records.

For many years the author has been engaged in the problems of evaluation of such diagrams. By means of an on-line computer connected in the system, the hydrographs can be converted into digital form, and the processor can evaluate them exactly, immediately, economically and with great efficiency, including the final report.

The co-ordinate reader can also be used as an off-line system and the computer is able to handle the perforated tapes of flow gauges. Analogue charts of any kind, e.g. from rainfall recorders, can be evaluated.

The control of a large system of reservoirs with fourteen storage basins for water supply of a world-renowned industrial area is planned using an extended computer for optimization of surplus quantities.

OBTENTION ET CONVERSION DES DONNEES ANALOGIQUES EN DONNEES DIGITALES A PARTIR DE LIMNIGRAMMES et D'HYETOGRAMMES ET TRAITEMENT PAR ORDINATEUR

Résumé. Le dépouillement manuel et individuel des diagrammes d'enregistrement des niveaux est un problème qui se pose tant du point de vue de la quantité que du point de vue de la qualité. Très souvent, le temps et le personnel font défaut pour le dépouillement de tous les diagrammes.

Depuis de nombreuses années l'auteur s'occupe des problèmes de dépouillement de ces diagrammes. Au moyen d'un calculateur, on procède à une conversion digitale, ce qui permet un calcul exact, rapide, économique et efficace allant jusqu'au stade final.

Le lecteur de coordonnées peut être utilisé comme une installation séparée, de même que l'ordinateur est capable de traiter les bandes perforées provenant des limnigraphes.

L'auteur cite comme exemple d'application pratique le contrôle d'un système de réservoirs d'eau comprenant 14 bassins pour l'alimentation en eau d'une grande région industrielle mondialement connue au moyen de calculs de prévisions quotidiens assurant une utilisation optimale des quantités d'eau disponibles.

CONVERSIÓN ANALÓGICA A DIGITAL Y OBTENCIÓN DE DATOS A PARTIR DE DIAGRAMAS DE REGISTRADORES DE NIVEL DE AGUA Y DE PRECIPITACIÓN, Y SU EVALUACIÓN POR MEDIO DE UN ORDENADOR CON PROGRAMAS VARIABLES

Resumen. La evaluación manual de datos de aforo es un gran problema cuantitativo también para una red de tamaño medio de estaciones. Hoy día es posible obtener los limnigramas y los diagramas por medio de un lector coordinado. Se convierten de analógicas a digitales por registradores de impulso. Un ordenador situado a continuación permite evaluar los impulsos de acuerdo con los programas establecidos.

Este procedimiento es rápido, preciso y económico. Sus posibilidades de utilización en hidrología y gestión de aguas son múltiples y su importancia para la planificación es esencial.

ПЕРЕВОД ДАННЫХ, ПОЛУЧАЕМЫХ ПО ГРАФИКАМ САМОПИСЦЕВ УРОВНЯ И ОСАДКОВ НА АНАЛОГО-ЦИФРОВУЮ СИСТЕМУ С ПОМОЩЬЮ СЧЕТНОЙ МАШИНЫ С ПЕРЕМЕННОЙ ПРОГРАММОЙ

Резюме. Ручная обработка лент самописцев уровня воды вследствие их огромного количества является сложной проблемой даже на сети средних размеров. В настоящее время можно обрабатывать ленты и диаграммы с помощью координатного регистратора. Они превращаются из аналоговых в цифровые с помощью импульсных регистраторов. Последующий компьютер учитывает импульсы в соответствии с программой. Этот метод является быстрым, точным и экономичным. Возможности его использования в гидрологии и в водном хозяйстве разнообразны и имеют существенное значение для планирования.

PURPOSE OF THE EVALUATION OF WATER GAUGE CHARTS

One of the main tasks of hydrology is the processing of data on water levels and discharges for practical water management as well as for statistical and basic scientific research. This work exclusively involves observations in the form of registration of water levels, i.e. hydrographs. These are made by recording gauges, the vertical and time scales of which may differ within the same network. Generally a vertical scale of 1 : 10 and a time scale of 2 mm/hr are chosen.

In the Federal Republic of Germany stage hydrographs are evaluated with respect to water level in cm and discharges in m^3/sec according to the regulations for preparation of the hydrological yearbooks.

DETERMINATIONS OF TERMS AND STATISTICAL PRINCIPLES

Table 1 shows the statistical and hydrological terms, as well as the fixed periods, for which the hydrographs are analysed. It distinguishes: principal values of the year under report; long term comparative values; extreme values.

The principal values are divided into arithmetic average values, ordinary and average boundary values.

In hydrology one generally uses daily values for the period 00.00 to 24.00 hr; in power generation and sanitary engineering other periods may be chosen.

The daily values are the arithmetic average values of water levels and discharges during one day, from which are derived the ordinary boundary values "NW" and "NQ" as well as the average boundary values "MNW" and "MNQ". For all other maximum flood values "HHW", "HHQ", and "HW", "HQ" exact time values are noted.

MANUAL EVALUATION OF PROGRESS LINES

The daily evaluation of water level recorder charts has been carried out manually. Where water levels have fluctuated during the day, the average value can be determined only by taking the arithmetic mean of a number of observations.

Evaluation of discharges in m^3/sec necessitates the use of a valid stage discharge curve. As this corresponds to a parabola with several parameters (due to the different

shapes of the discharge profiles and to the roughness of the profile walls), the average daily discharge cannot be derived from the daily average level. Instead discharge values corresponding to stage values at different times of the day must be averaged. Thus the evaluation of water level recorder sheets becomes a great quantitative problem.

According to general experience, one assistant is necessary for the evaluation of twenty water gauges and the statistical processing of the data and preparation of the diagrams. As this work is not very interesting and the evaluators are engaged on additional work, the evaluation is not carried out as exactly and as carefully as necessary.

REQUIREMENTS OF AN EVALUATION DEVICE

Since in measurement technology analogue to digital conversion of records is important, suitable diagram readers were developed a few years ago and used in the United States and Federal Republic of Germany. The charts are traced manually so that even faded, dirty or damaged diagrams can be evaluated. The specification of the chart readers includes:

1. It should resolve a trace into pairs of co-ordinates expressed in digital form.
2. The resolution and the reproducible accuracy must be up to 0.1 mm.
3. The origin of co-ordinates must be adjustable.
4. The sign of the measuring points must be determined.
5. The step distance in the x axis must be selectable from 0.1 mm to 10.0 mm.
6. Mirror-inverted level lines of write levels with reversing spindles or level lines with basic translation of hunting write levels are determined according to the effective y values.
7. Time inaccuracies due to the recording drum clock-work and subsequent changes in length of paper transported must be compensated for by establishing the nominal/actual factor of every record before scanning.
8. Subsequently the computer should be able to evaluate traces from stored programmes suitable for all kinds of diagrams, then the computer should deliver clear texts and punched tapes.

THE OPERATING PRINCIPLE OF THE DIAGRAM READER

Since 1966 diagram readers have been developed, manufactured and offered at reasonable prices in the Federal Republic of Germany. Such a device must determine the co-ordinates x_a, y_a and x_b, y_b of the line $a - b = s$ on a graphical copy. It must be able to operate in the four quadrants of the Cartesian system of co-ordinates. In addition to the absolute values x and y, the signs and the direction of motion of the positioning control must be distinguished, stored and read out. For technical reasons the incremental method of trunk measurement is used; the analogue to digital conversion is performed by angular incremental encoders with the following pulse counters. In cases of power failure the trunk information may get lost; this is not really a disadvantage since, by finding the selected and marked origin of co-ordinates, a connexion can be made to previous measurements.

The trunk to be measured in the diagram is scanned by a scanning device, the positioning control. Its motions in the x and y axes are transferred, via steel band deflections, to two rotary angular pulse generators. The trunk scanned photo-

electrically is divided into sections Δs of the same size according to x and y values. Every section applied corresponding pulses to the following x and y counters. These counters must indicate the direction of motion in addition to the number of pulses equivalent to the analogue value. The number of pulses is indicated by two pulse sequences, either leading or trailing, one of which corresponds to a pulse signal, the other to a scanning device divided by $d = \Delta S/4 = 90^\circ$ el. A directional discriminator evaluates both signals and controls the counting direction reversing correspondingly in a forward reverse counter (Fig. 1).

The forward-reverse counters count the pulses and give a digital display in an off-line transmission. At the same time the counters store the digits or deliver them, via following devices, in clear text, on cards, punched tape or magnetic tape.

The values may also be applied directly to an on-line stored programme computer for immediate evaluation. In order to avoid the relatively long procedures of data delivery and data feeding, the author has already suggested this combination of diagram reader and computer for a reading device which was purchased by the Ruhrtalsperrenverein, Essen. This method is more economical because no punches and readers are required; it is more simple and safer in operation since there is no information carrier in between.

The number of places in the digital display depends on the resolution and size of the diagram. Normally four-figure, and sometimes even five-figure, digits require 0.1 mm resolution, 1 × 1 scale evaluation and 9999.9 mm scanning length. The data delivery corresponds to the digital display; it has a binary coded decimal notation.

CURVE PROCESSING BY MEANS OF A COMPUTER

Any usual memory programmed computer may be linked on-line to the diagram reader. It should, of course, have the necessary flexibility, with interrupt levels and the desirable peripherals.

As practical experience has shown, a computer with 4K capacity and an 18-bit word length, having two programme levels with two accumulators and four index registers each, is sufficient. Data input, as well as programme loading, is done by means of an eight-channel tape reader.

The computer may be used as an off-line processor, too, handling gauge information by the same programme.

PROCEDURE FOR EVALUATION OF HYDROGRAPHS AND DIAGRAMS

The following block diagrams show the systematic process of the evaluation of water level recorder charts: Figure 2: an evaluation device as on-line unit; Figure 3: the daily and monthly evaluations of level lines in order to obtain major values; Figure 4: the evaluation of quarters of year, half-years, discharge years and series of years according to the official regulations.

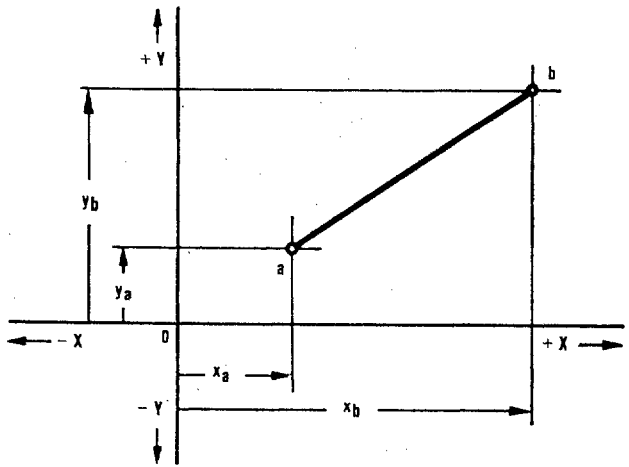
According to our experience, 40-60 min are required for reading of the monthly five or six water level recorder charts and the parallel evaluation in the computer, including the final report in the text in clear and on perforated tapes, corresponding to the course of the hydrographs. For the twelve months of a discharge year, this would amount to about 12 hr, although practically they are evaluated each month when they are received.

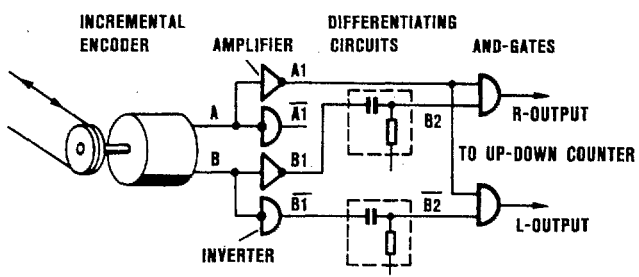
The evaluation of the monthly perforated tapes for quarters, half-years, discharge

Table 1 — *Comments on hydrological main data and their abbreviations*

	Water stage in cm W	Discharge in m^3/s Q	Discharge per unit area in $l/s\ km^2$ q	Water temperature in °C T	Explanations
a. Highest known value (HH) (absolute maximum)	HHW	HHQ	HHq	HHT	Highest value known up to now, e. g., HHQ is the highest discharge ever measured at the station concerned.
b. Highest value (H) of the period under consideration (maximum)	HW	HQ	Hq	HT	Maximum value for a specified period. Therefore the highest value (H) always stands together with a time specification. This may be a coherent period (month, year) or a group of years. But it may refer to single months of a year-group as well. E. g. HW 1956/65 is the highest water stage measured during these years, and I HW 1956/65 the highest water stage encountered in the months of January of the years 1956—1965.

c. Mean highest value (MH) of similar sections of the period under consideration (mean maximum)	MHW	MHQ	MHq	MHT	Arithmetic mean of the highest values (H as specified under b) of several successive years. E. g., MHQ 1931/60 is the mean of the highest discharges of each year of this 30 years group. If MHQ 1931/60, on the other hand, is obtained by determining the highest discharges measured in February of each of the 30 years and by averaging these 30 discharge values.
d. Mean value (M) of period under consideration	MW	MQ	Mq	MT	Arithmetic mean of all main observations (daily values) during period under consideration. For time specification see b. above. For periods of up to one year this value is determined as arithmetic mean of all main observations; for a period of several years from the mean values of the respective months or years (mean of the means).
e. Mean lowest value (MN) of similar sections of the period under consideration (mean minimum)	MNW	MNQ	MNq	MNT	The explanations given under c. apply analogously. For gauges of which daily means or approximations are published, MW, MQ or MT denote the lowest daily mean and, respectively, the lowest approximation thereof.
f. Lowest value (N) of the period under consideration (minimum)	NW	NQ	Nq	NT	The explanations, given under b. above, apply analogously. For daily means, see e. above.
g. Lowest known value (NN)	NNW	NNQ	NNq	NNT	The explanations given under a. and e. (second sentence) apply analogously.





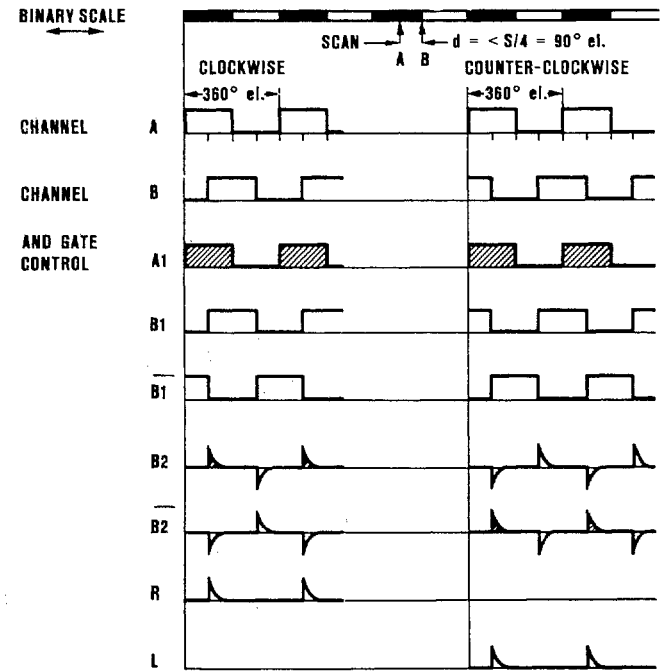


Figure 1 – *Block diagram of displacement transducer with direction discriminator*

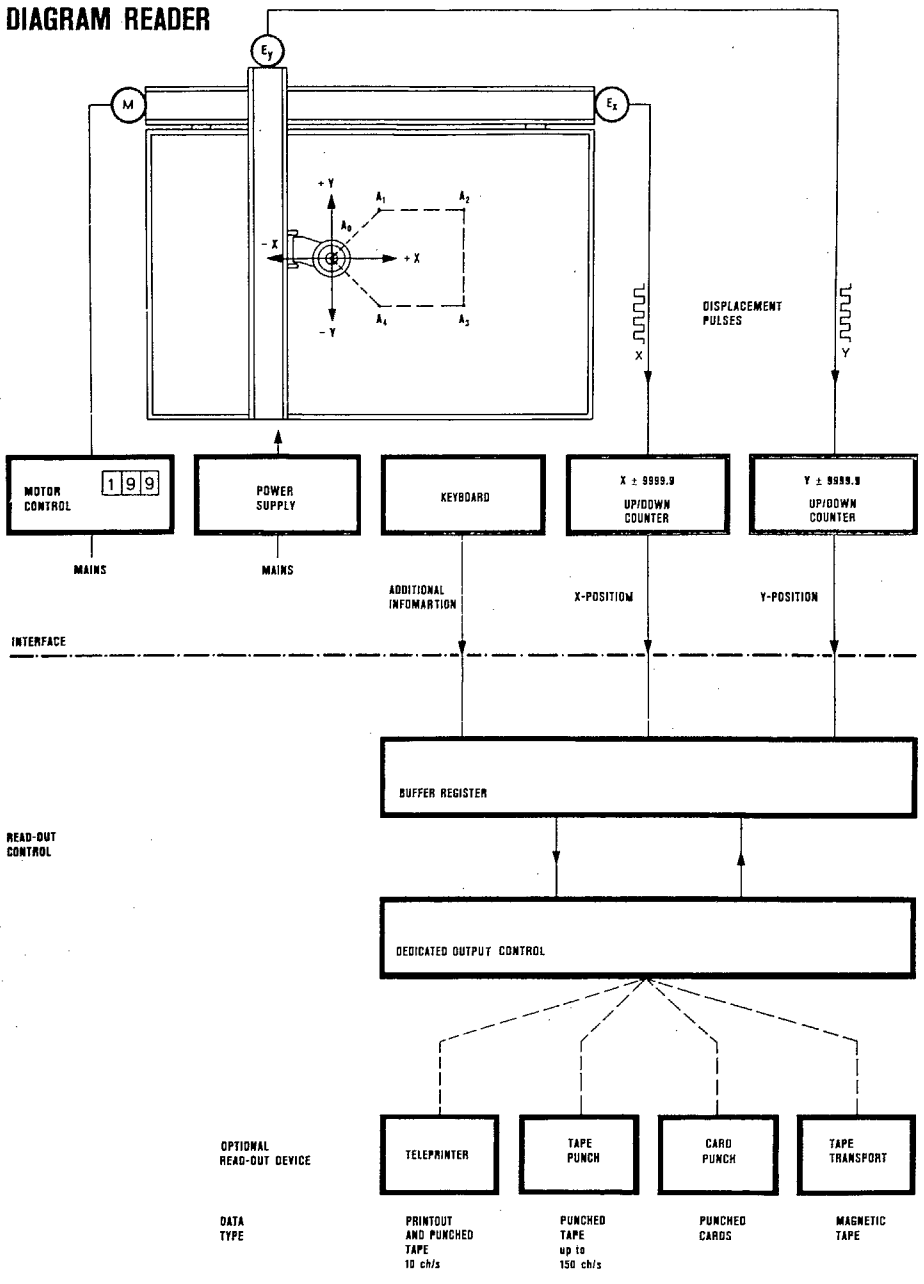


Figure 2 – *Data acquisition of two-dimensional diagrams*

years and series of years, as well as discharge quantities and discharge heights and flow duration values, takes 1/2 to 1 hr.

Thus from one to two continuous working days are necessary for the annual evalution of a water gauge, including reading of the sheets up to the final report.

Considering the purchase price for an evaluation device of DM.100,000 to DM.120,000, amortization can be achieved within 3 1/2 to 4 years with annual evaluation of from forty to fifty water gauges, by saving of working time compared with manual evaluation. At operation periods exceeding 8 hr per day the profitability of the device correspondingly increases.

In this case a 0.4 mm sequence of steps in the x axis has been selected with 2 mm/hr transport of the level drum. Thus five scannings (one scanning every 12 min) corresponds to one hour of the level. This density of evaluation and the corresponding accuracy would not be achieved with the present manual method.

Evaluation with 0.1 mm step distance, which corresponds to one scanning every 3 min, is possible, but has no practical importance since longer evaluation periods would be required and thus the total output decreased.

APPLICATIONS OF THE EVALUATION DEVICE

There are many possibilities for the use of this evaluation device: (a) the evaluation of discharge measurements according to the procedure of Caesperlein; (b) the preparation of discharge tables based on discharge measurements; and (c) the evaluation of tapes from rainfall recorders according to the method of Haeuser/Reinhold with respect to rainfall intensity, rainfall quantities and periods. This will be carried out according to a programme yet to be devised, and will be graphically recorded by a subsequent plotting apparatus. This would enable determination of practical values for planning and calculation of canalizations, sewage treatment plants, provided the existence of a network of rainfall recorders operated for a longer period.

As a final aim it is planned to manage the total river area of the River Ruhr, a tributary of the Rhine, extending over 4,500 km^2 and to control a system of fourteen reservoirs with a total storage capacity of 471 hm^3. With annual extraction of 1.2 milliards m^3 for the water supply of a population of 5 million and most of the North-Rhine-Westphalian industry, with an annual average discharge of 2.4 milliards m^3 fluctuating up to $\pm$ 40 per cent, control is only possible by exact water management.

The control of the Ruhr reservoirs is effected according to the uninfluenced discharge of the Ruhr into the Rhine. The contributions from the reservoirs must be discharged 3-4 days previously. This results in an average loss of 25 per cent of the necessary surplus quantity. Calculation in advance of the uninfluenced flow at the mouth from the inflow into the headwater of the Ruhr, the precipitation and discharges in the whole area and other factors and parameters might enable a reduction of this factor to one half or less and would save 50 million m^3 and more in one period for better use.

Furthermore, a more exact determination of the uninfluenced discharge heights would result in increased storage in the reservoirs and thus increased contributions. In this example the computer would be used as a kind of process calculator for continuous determination, in advance, of the flow at the month and for timely operation of the reservoirs.

There are many other examples for use in water management proving the versatility, accuracy and speed of the evaluation device; besides which it is very profitable.

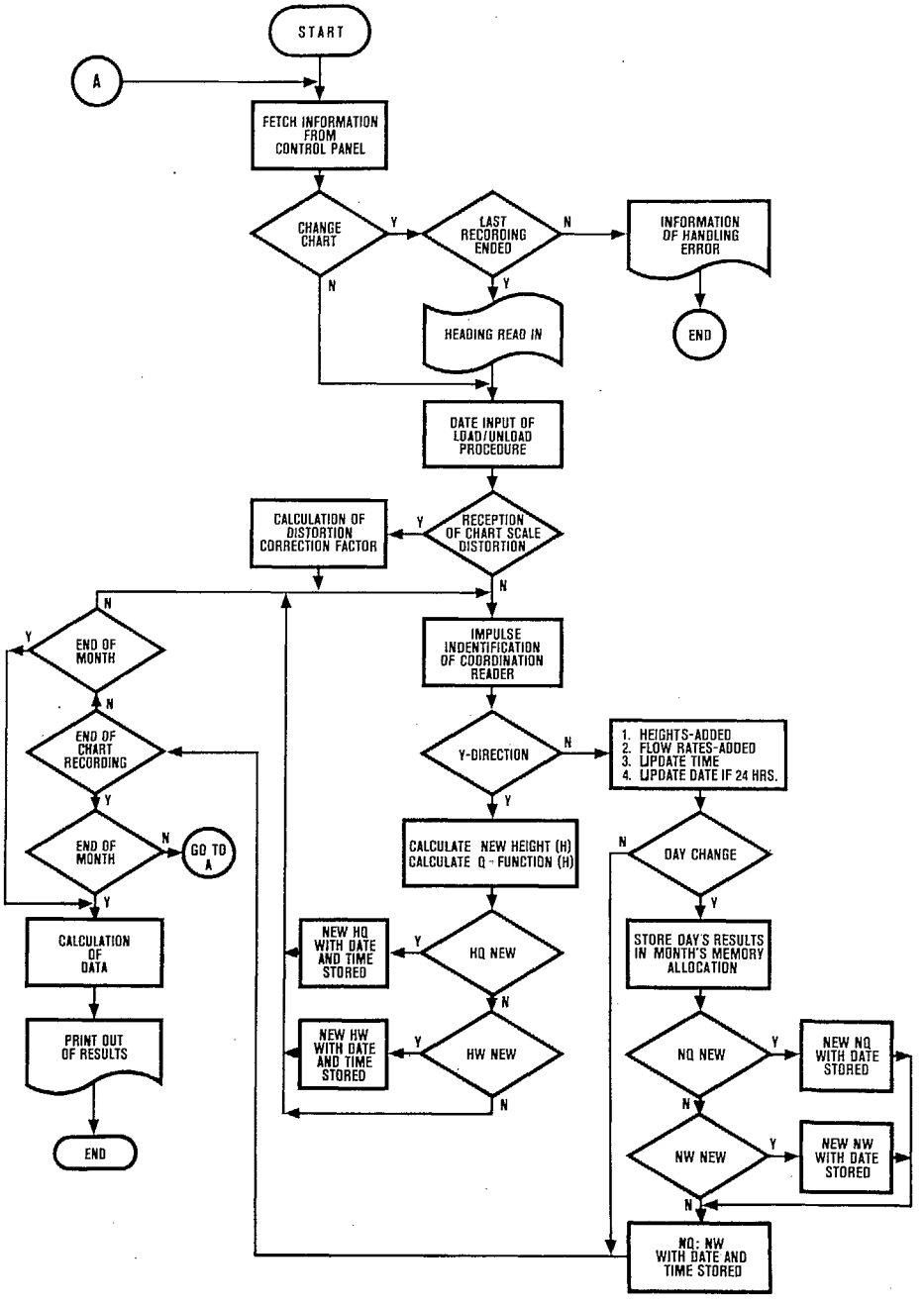


Figure 3 – *Monthly evaluation*

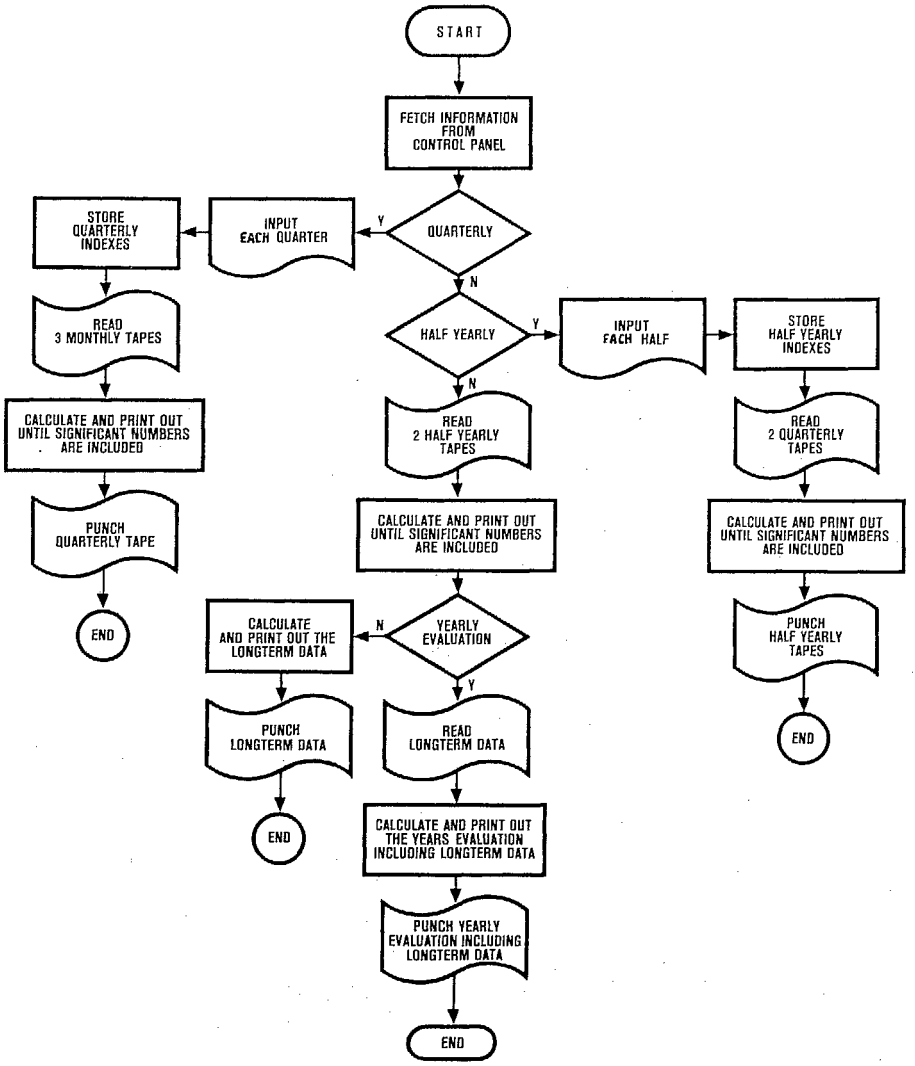


Figure 4 – *Annual evaluation*

DISCUSSION

Bliessner: When you correct the sheet with regard to the time units (changes of the length of the sheet)? Can you also correct a sloping sheet?

Kleinenbroich: We fix the sheet precisely on the parallel line on the reader. You can see then whether the sheet had operated exactly.

Manual and automatic evaluation of hydrometric data in Israel

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Abstract. In 1948, when the State of Israel was established, the Hydrological Service was operating twenty-five hydrometric stations. Since then, new water-level recording instruments have been introduced in some 120 hydrometric stations. This network is continuously growing and, consequently, so is the volume of field data. In the past, manual methods were satisfactory; it was possible to make all computations by hand and issue the hydrological information promptly. At present, however, this task seems to be almost impossible, and calls for automatic processing systems.

The paper consists of two main parts: the first one describes the conventional manual methods of computation of hydrological data, as applied in the Israeli Hydrological Service until recently. The second part describes a newly developed system for automatic processing of hydrometric data. This Levy-Graphicomp system is an intergrated software-hardware graphic system based on an analytical approach. It is a real-time graphic system consisting of a curve follower, processor, display unit, and a teletype for input-output. A plotter for graphic output is optional. This system enables a direct and processing of hydrological data in the office in a fraction of the time required by the conventional means.

CALCUL MANUEL ET AUTOMATIQUE DES DONNEES HYDROMETRIQUES EN ISRAEL

Résumé. En 1948, lors de l'établissement de l'Etat d'Israël, le Service hydrologique opérait sur 25 stations hydrométriques. Depuis, de nouveaux limnigraphes ont été placés en quelque 120 stations. Ce réseau se développe de façon continue et, en conséquence, le volume des données s'accroît. Dans le passé, les méthodes manuelles donnaient satisfaction ; il était possible de faire tous les calculs à la main et de fournir en temps utile les informations hydrologiques. Aujourd'hui, ce travail devient presque impossible et il faut faire appel à des systèmes automatiques de traitement des informations.

Cette note comprend deux parties : la première décrit les méthodes manuelles conventionnelles de calcul des données hydrologiques utilisées par le Service hydrologique d'Israël dans un passé récent ; la seconde décrit le système nouvellement mis en place pour le traitement automatique des données hydrologiques. Ce système, appelé Levy-Graphicomp, est un système graphique intégré "software-hardware" basé sur une approche analytique complète. C'est un système graphique en temps vrai comportant un suiveur de courbe, un reproducteur, une unité d'affichage et un télétype pour entrée-sortie. Un suiveur du graphique de sortie est facultatif.

Ce système permet une analyse directe et un traitement des données hydrologiques au bureau dans une fraction de temps réduite en comparaison des moyens conventionnels.

EVALUACIÓN MANUAL Y AUTOMÁTICA DE DATOS HIDROMÉTRICOS EN ISRAEL

Resumen. En 1948, cuando se estableció el Estado de Israel, el Servicio Hidrológico mantenía en funcionamiento 25 estaciones hidrométricas. Desde entonces se han introducido nuevos instrumentos de registro de nivel en 120 estaciones hidrométricas. Esta red está en crecimiento constante, y en consecuencia, también crece el volumen de datos de campo. En el pasado, los métodos manuales resultaban satisfactorios ; era posible hacer mano a mano todos los cálculos, y editar la información hidrológica a tiempo. Actualmente, sin embargo, esta tarea resulta casi imposible y exige el empleo de sistemas automáticos de proceso de datos.

La comunicación presenta dos partes principales; la primera describe los métodos manuales convencionales para el cálculo de datos hidrológicos, tal como se aplicaba hasta hace poco por el

Servicio Hidrológico de Israel. La segunda parte describe el sistema puesto a punto recientemente para el proceso automático de datos hidrométricos. Este sistema, llamado Levy-Graphicomp. es un sistema gráfico integrado softlware-hardware basado en un enfoque general analítico. Es un sistema gráfico de tiempo real que consta de seguidor de curva, procesador, unidad de despliegue y teletipo para entrada-salida. Como opcional figura un trazador para representación gráfica de salida.

El sistema permite un análisis directo y un proceso de datos hidrológicos en una fracción del tiempo exigido por medios convencionales.

РУЧНАЯ И АВТОМАТИЧЕСКАЯ ОБРАБОТКА ГИДРОМЕТРИЧЕСКИХ ДАННЫХ В ИЗРАИЛЕ

Резюме. В 1948 г., когда было основано государство Израиль, в подчинении гидрологической службы находилось 25 гидрометрических станций. С того времени на приблизительно 120 гидрометрических станциях введено в строй новое оборудование для регистрации уровней воды. Эта сеть непрерывно растет, и соответственно растет объем полевых данных. В прошлом ручные методы были удовлетворительными; было возможным сделать все расчеты вручную и во-время выпустить гидрологическую информацию. В настоящее время, такая задача является почти невозможной и требует автоматической системы обработки.

Доклад состоит из двух основных частей; в первой части описываются обычные ручные способы обработки гидрологических данных, применяшиеся гидрологической службой Израиля до недавнего времени. Во второй части описывается недавно разработанная системя для автоматической обработки гидрометрических данных. Эта система, названная Леви-Графикомп, является графической интегрирующей системой, основанной полностью на аналитическом подходе. Это современная графическая система, состоямая из прослеживателя кривых, устройства для обработки данных, показывающего устройства и телетайпа для ввода и выхода информации. Устройство для выдачи данных в графической форме устанавливается по желанию.

Эта система дает возможность проводить непосредственные анализы и обработку гидрологических данных в помещении службы за часть того времени, которое требовалось при применении стандартных методов.

PART I. MANUAL EVALUATION OF HYDROMETRIC DATA

1. Introduction

In 1948, when the State of Israel was established, the Hydrological Service was operating twenty-five hydrometric stations on rivers and wadis. Since then, new instruments for water-level recording have been developed and introduced in some 120 hydrometric stations. Some of the measuring sites are situated in remote places in the arid zone of the south which can be visited only occasionally. The water-level recorders were, at first, drum-type, with mechanical clocks working for periods up to two months. Later, a strip-chart recorder, with an electrical clock working unattended for a period up to twelve months, was developed and installed in the majority of hydrometric stations in Israel.

During the last twenty years many water-level recorders have been installed near measuring weirs on springs, as well as instruments recording fluctuations of ground water in wells.

In the future, more hydrometrical stations will be added to the network existing today to procure data needed for national development plans. With the expansion of

the network, the volume of hydrometric data grows. The accumulation of field data brings in its train the problem of handling the growing amount of data. While in the past it was possible to make all the computations manually and issue quickly the hydrological information required by engineers and planners, this task is almost impossible at present and calls for other means. The Hydrological Service in Israel, therefore, decided to introduce automatic means for evaluation of the hydrological data. Three possibilities were considered :

1. To introduce digital water-level recorders, the output of which is a punched tape, which can go directly to a computer;
2. To build a chart-reading instrument, the output of which being a punched tape which is taken to a computer for data processing;
3. To develop and build special-purpose equipment in which the chart-reading and computation will be done at the same time in the office of the Hydrological Service.

Examining these three possibilities the third one was chosen. In this paper, the authors explain the stream-gauging and computation procedure in Israel and describe the special-purpose equipment developed for automatic processing of the hydrometric data.

2. *Manual evaluation of hydrometric data*

The manual evaluation of the yearly flow at a measuring site is based on the following data and computation procedure:

1. Current-meters are calibrated in the Technical Institute in Haifa and calibration tables are prepared.
2. With the aid of these calibration tables the discharge measured at different water levels is computed.
3. The computed discharge values are plotted as a function of water level. Then, a best fitting curve is drawn through these points, giving the stage-discharge curve.
4. A stage-discharge table is computed from this curve.
5. The water-level recorder chart (the limnigraph) is then prepared for reading ; the base line is divided into days (24-hr intervals) by drawing parallel lines perpendicular to the base-line. The exact time scale is computed from time-marks made on the chart by the observer on his visit to the measuring site.
6. In case of a drum-type water-level recorder, additional calculations of the stepped base line elevation are done (see paragraph 6 (c)); the same applies to a reversed chart (see paragraph 6 (d)).
7. The manual reading of the chart is as follows: water levels and the corresponding time of chosen points on the limnigraph are tablulated. Corresponding discharges are computed from the stage-discharge table and are tabulated.
8. A hydrograph is plotted for the observation period, the horizontal axis being the time and the vertical the discharge.
9. The computation of daily discharges is done by planimetering the areas between the zero-flow line and the hydrograph, limited by the 24-hr verticals at the end of each day.
10. Those daily discharges are tabulated totals for each flood. Totals of monthly and yearly flow are computed as well.

This computation procedure is time-consuming and tedious.

3. *Current meter calibration*

Current meter calibration is performed in the Hydraulic Laboratory of the Technical Institute in Haifa. It is done by means of a rating-platform, which moves the

current-meter at a constant velocity along the still water in the rating basin. Table 1 shows the calibration data of current meter No. 304. The results of eighteen tests at

Table 1

Current Meter No. 304

Date of calibration: 13.3.68
Lab. Certificate No. 3316

$\Delta V_0 = 0.9240$

$\Delta n = 11.180$

$V_0 = \frac{\Delta V_0}{\Delta n} n = \frac{0.9240}{11.180} n$

$= 0.08265\, n$

$\Delta V = 0.00227 n + 0.01538 ; \quad n < 6.0$
$\Delta V = -0.00510 n + 0.05960 ; \quad n > 6.0$

$V = 0.08492 n + 0.01538 ; \quad n < 6.0$
$V = 0.07755 n + 0.05960 ; \quad n > 6.0$

Test No.	V' measured m/sec	n revolutions per second	V_0 0.08265 n m/sec	ΔV $V' - V_0$	V computed m/sec	$\Delta V'$ +	$\Delta V'$ −	Remarks
1	0.0483	0.336	0.0278	+0.0205	0.0439		0.0044	
2	0.0883	0.858	0.0709	+0.0174	0.0882		0.0001	
3	0.1390	1.465	0.1211	+0.0179	0.1398	0.0008		
4	0.1465	1.561	0.1290	+0.0175	0.1479	0.0014		
5	0.2400	2.658	0.2197	+0.0203	0.2411	0.0011		
6	0.3420	3.846	0.3179	+0.0241	0.3420	—	—	
7	0.4120	4.663	0.3854	+0.0266	0.4114		0.0006	
8	0.4760	5.443	0.4499	+0.0261	0.4776	0.0016		
9	0.6200	7.229	0.5975	+0.0225	0.6202	0.0002		
10	0.8260	9.882	0.8167	+0.0093	0.8259		0.0001	
11	0.9240	11.180	0.9240	±0.0000	0.9266	0.0026		
12	1.0050	12.097	0.9998	+0.0052	0.9977		0.0073	
13	1.1350	13.793	1.1400	-0.0050	1.1292		0.0058	
14	1.2500	15.385	1.2716	-0.0216	1.2527	0.0027		
15	1.3200	16.271	1.3448	-0.0248	1.3214	0.0014		
16	1.4900	18.261	1.5093	-0.0193	1.4757		0.0143	Cancelled
17	1.6500	20.454	1.6905	-0.0405	1.6458		0.0042	
18	0.1090	1.194	0.0987	+0.0103	0.1168	0.0078		

Table 2

Current Meter No. 304

Date of calibration: 13.3.68
Lab. certificate No. 3316

$v = 0.08492\,n + 0.01538 \quad n < 6.0$
$v = 0.07755\,n + 0.05960 \quad n > 6.0$

n revs. per second	v m/sec										n revs. per second
	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	
0					0.049	0.058	0.066	0.075	0.083	0.092	0
1	0.100	0.109	0.117	0.126	0.134	0.143	0.151	0.160	0.168	0.177	1
2	0.185	0.194	0.202	0.211	0.219	0.228	0.236	0.245	0.253	0.262	2
3	0.270	0.279	0.287	0.296	0.304	0.313	0.321	0.330	0.338	0.347	3
4	0.355	0.364	0.372	0.381	0.389	0.398	0.406	0.415	0.423	0.431	4
5	0.440	0.448	0.457	0.465	0.474	0.482	0.491	0.499	0.508	0.516	5
6	0.525	0.533	0.540	0.548	0.556	0.564	0.571	0.579	0.587	0.595	6
7	0.602	0.610	0.618	0.626	0.633	0.641	0.649	0.657	0.664	0.672	7
8	0.680	0.688	0.696	0.703	0.711	0.719	0.727	0.734	0.742	0.750	8
9	0.758	0.765	0.773	0.781	0.789	0.796	0.804	0.812	0.820	0.827	9
10	0.835	0.843	0.851	0.858	0.866	0.874	0.882	0.889	0.897	0.905	10
11	0.913	0.920	0.928	0.936	0.944	0.951	0.959	0.967	0.975	0.982	11
12	0.990	0.998	1.006	1.013	1.021	1.029	1.037	1.044	1.052	1.060	12
13	1.068	1.076	1.083	1.091	1.099	1.107	1.114	1.122	1.130	1.138	13
14	1.145	1.153	1.161	1.169	1.176	1.184	1.192	1.200	1.207	1.215	14
15	1.223	1.231	1.238	1.246	1.254	1.262	1.269	1.277	1.285	1.293	15
16	1.300	1.308	1.316	1.324	1.331	1.339	1.347	1.355	1.362	1.370	16
17	1.378	1.386	1.393	1.401	1.409	1.417	1.424	1.432	1.440	1.448	17
18	1.456	1.463	1.471	1.479	1.487	1.494	1.502	1.510	1.518	1.525	18
19	1.533	1.541	1.549	1.556	1.564	1.572	1.580	1.587	1.595	1.603	19
20	1.611	1.618	1.626	1.634	1.642	1.649					20

	7	8	9
1	1	1	1
2	1	2	2
3	2	2	3
4	3	3	4
5	4	4	4
6	4	5	5
7	5	6	6
8	6	6	7
9	6	7	8

different velocities performed during the calibration on 13 March 1960 are drawn on Figure 1. A straight line is drawn through the origin and one of the calibration test points, in this example giving a line the equation of which is $V_0 = 0.08265/n$. For each of the test points V_0 is computed using measured n and the differences in corresponding velocities between the measured velocity V and computed $V_0(\Delta V = V_1 - V_0)$. These differences are drawn to scale (see Figure 1, round points). Through those points the best-fitting straight lines are drawn giving, in this particular case, two lines crossing at a point which corresponds to 6 revs per sec. The computed velocity from this calibration is

$$V = V_0 + \Delta V$$

giving two straight lines, the equation of which is

$$V = 0.08492\,n + 0.01538 \text{ for } n < 6 \text{ revs per sec}$$
$$V = 0.07755\,n + 0.05960 \text{ for } n > 6 \text{ revs per sec.}$$

Using those equations a rating table is prepared. This may be like the rating table in Table 2, giving velocities for any number of revs per sec, or a table giving velocities for a certain number of revolutions and measured time (e.g. 60 revs in 40, 41, 42 etc. sec).

4. Current meter measurements

The discharge measurement is usually performed by the two-point method (velocity measured at 0.2 and 0.8 of the depth of each vertical – the mean of these two velocities being the mean velocity in vertical). Only in exceptional cases is the multiple-point method of discharge measurement done. Sample measurements are shown in Figures 2 and 3.

5. Stage-discharge rating curve

Figure 4 shows an example of a stage-discharge curve for the Nahal Ayalon measuring station. There is a low-water control consisting of a concrete weir built between abutments of an old bridge. The weir becomes totally submerged at discharges larger than 20 m^3/sec. It was calibrated by current meter and float measurements.

This stage-discharge curve may be drawn on logarithmic paper and a straight line drawn through the plotted points gives the discharge formula $Q = a + b\,.\,(H - H_0)^n$.

6. The hydrograph

Figure 5 shows the hydrograph of the Nahal Ayalon for the flood of 18-25 January 1967. Curve H shows the fluctuations of water level as recorded by the automatic water-level recorder. Curve Q is the corresponding hydrograph.

On this graphical representation of the flood, all the data gathered in the field and results of measurements are plotted. This procedure allows a quick check of the limnigraph and hydrograph, and gross errors in plotting and computation may be avoided.

Many methods may be used in order to draw this hydrograph (see paragraph 2 above). A method for quick evaluation of flow from the limnigraph is explained in Figure 6. On the plotted stage-discharge curve we find, for the water level H_1, the corresponding discharge Q_1, which is expressed by the length of the abscissa OA_1. Draw a line at a 45° angle to the abscissa through the head of co-ordinates O. Point

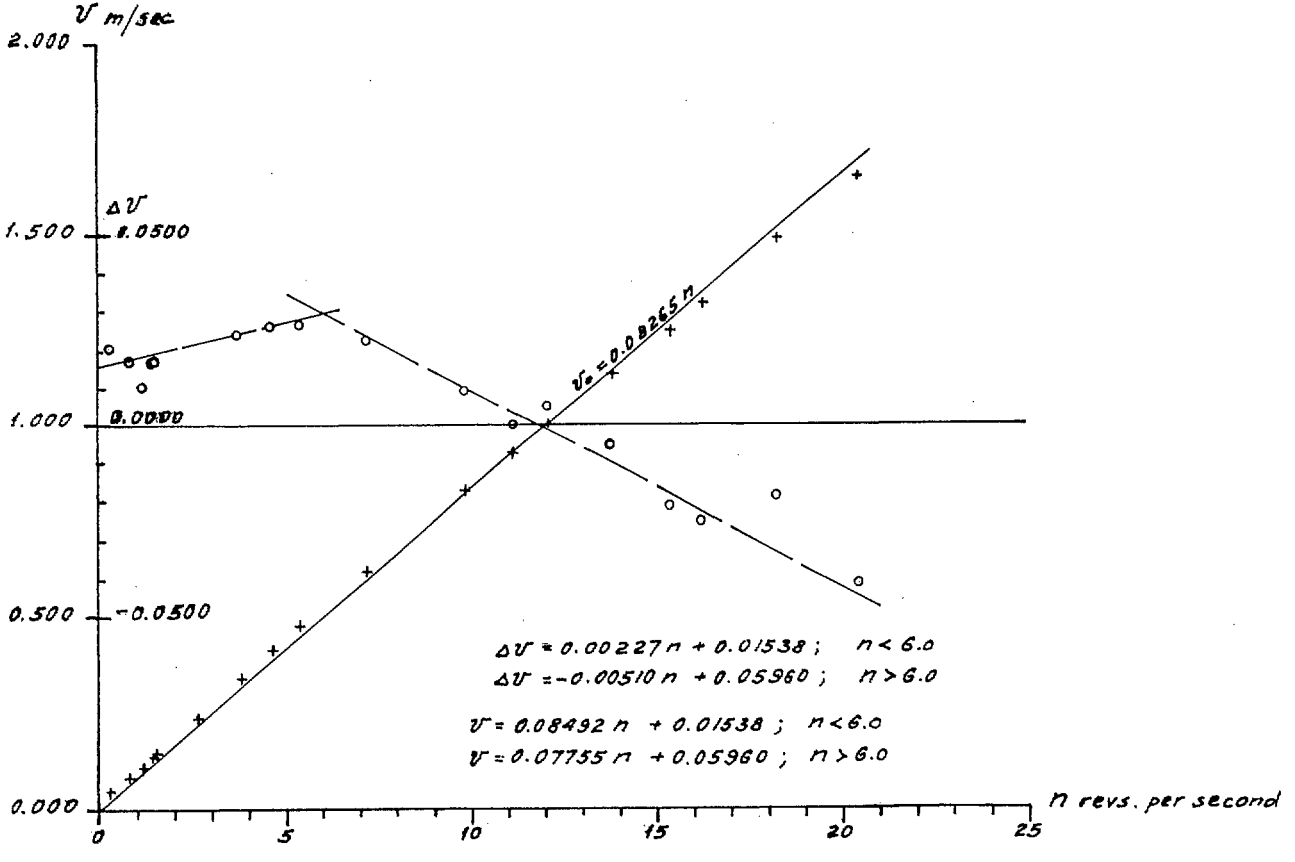


Figure 1 – *Results of 18 tests during calibration of current meter 304*

Ministry of Agriculture
Water Commission
Hydrological Service

District: Central
Gauging Station: N. AYALON at Beit Dagan - Yehud Rd. Co-ordinates: East 136.05 North 158.19
Source of Water: Storm run-off

Meas. No. 13

Current Meter					Stop Watch No.	Weather		Colour of water
Date and hour	Type and No.	Date of calibration	Signal after every	Kind of susp.			Wind	
18.1.67 13³⁰-14⁰⁰	N.S. 625	June 1964	30 revolutions	cable	51		in direction / against direction } of flow; no wind	

Bottom covered by	Vegetation in stream bed	Kind of flow
boulders / gravel / sand / mud / concrete / stone paving	bushes; grass; no vegetation	calm; turbulent; disturbed by

Water sample taken / not taken Bottle No. 73

Remarks on progress of measurement, disturbances, changes in stream bed etc.:

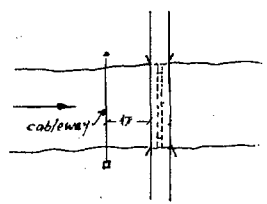
1. Very good conditions for discharge measurement.
2. No changes in stream channel.
3. Intake pipe clean

Surface float measurement
Length of stretch: l = 20.00 m
Time: 13.8 seconds
14.0 "
13.9 "
13.8 "
14.1 "
Average 13.9 seconds

Maximum surface velocity:
$\max V_o$ = 1.439 m/sec

Results of measurement
Average W.L.: +22.66 m
Wetted section: 26.76 m²
Discharge: 26.2 m³/sec
to add or to subtract
Total discharge: 26.2 m³/sec
Mean velocity in section: V_m 0.978 m/sec
$\varphi = \frac{V_m}{\max V_o}$ = 0.68

Sketch of site



Measured by: H. Sobol
Computed by: A. Willner Date: 7.2.67
Checked by: M. Zvi Date: 10.2.67

Figure 2 – *Computation of current meter discharge measurement*

Ministry of Agriculture
Water Commission
Hydrological Service

Gauging Station: N. AYALON at Beit Dogan - Yehud Rd. Date: 18.1.67 Meas. No. 13 Sheet No. 1 of 1 sheets

Hour	Distance from initial point m	Gauge Reading: In measuring section m	Gauge Reading: at W.L. Recorder m	Depth: of Water m	Depth: of observation m	Revolutions m	Time in seconds	Revolutions per second	Velocity: at point v m/sec	Velocity: mean in vertical v_m m/sec	Velocity: mean between verticals V_m m/sec	Area a m^2	$q = V_m \times a$ m^3/sec
13[30]	11.85	+22.65	+22.65	0.00	R.E.W.	zero	velocity		0.000	0.000			
											0.146	0.25	0.036
	13.00			0.43	0.26	60	42.5	1.41	0.292	0.292			
											0.633	2.18	1.380
	15.00			1.75	0.35	180	32.3	5.57	1.155	0.975			
	"			"	1.40	120	31.2	3.84	0.796				
											1.161	4.08	4.737
	17.00			2.33	0.47	240	37.2	6.45	1.338	1.348			
	"			"	1.86	240	36.6	6.55	1.359				
											1.309	4.68	6.126
	19.00			2.35	0.47	240	33.0	7.27	1.505	1.271			
	"			"	1.88	180	36.0	5.00	1.037				
											1.204	4.48	5.394
	21.00			2.18	0.44	240	40.1	5.98	1.240	1.138			
	"			"	1.74	240	48.0	5.00	1.037				
											1.086	4.34	4.713
	23.00			2.16	0.43	240	46.6	5.15	1.068	1.035			
	"			"	1.73	180	37.2	4.83	1.002				
											0.814	3.93	3.199
	25.00			1.77	0.35	120	36.6	3.27	0.678	0.593			
	"			"	1.42	120	49.8	2.45	0.508				
											0.389	1.46	0.568
	26.00			1.15	0.23	60	76.8	0.78	0.163	0.185			
	"			"	0.92	60	60.0	1.00	0.207				
											0.092	0.89	0.082
	27.00			0.63	zero	velocity			0.000	0.000			
												0.47	
14[00]	28.50	+22.66	+22.66	0.00	L.E.W.								
												26.76	26.235
													≈ 26.2

Figure 3 – *Computation of current meter discharge measurement.*

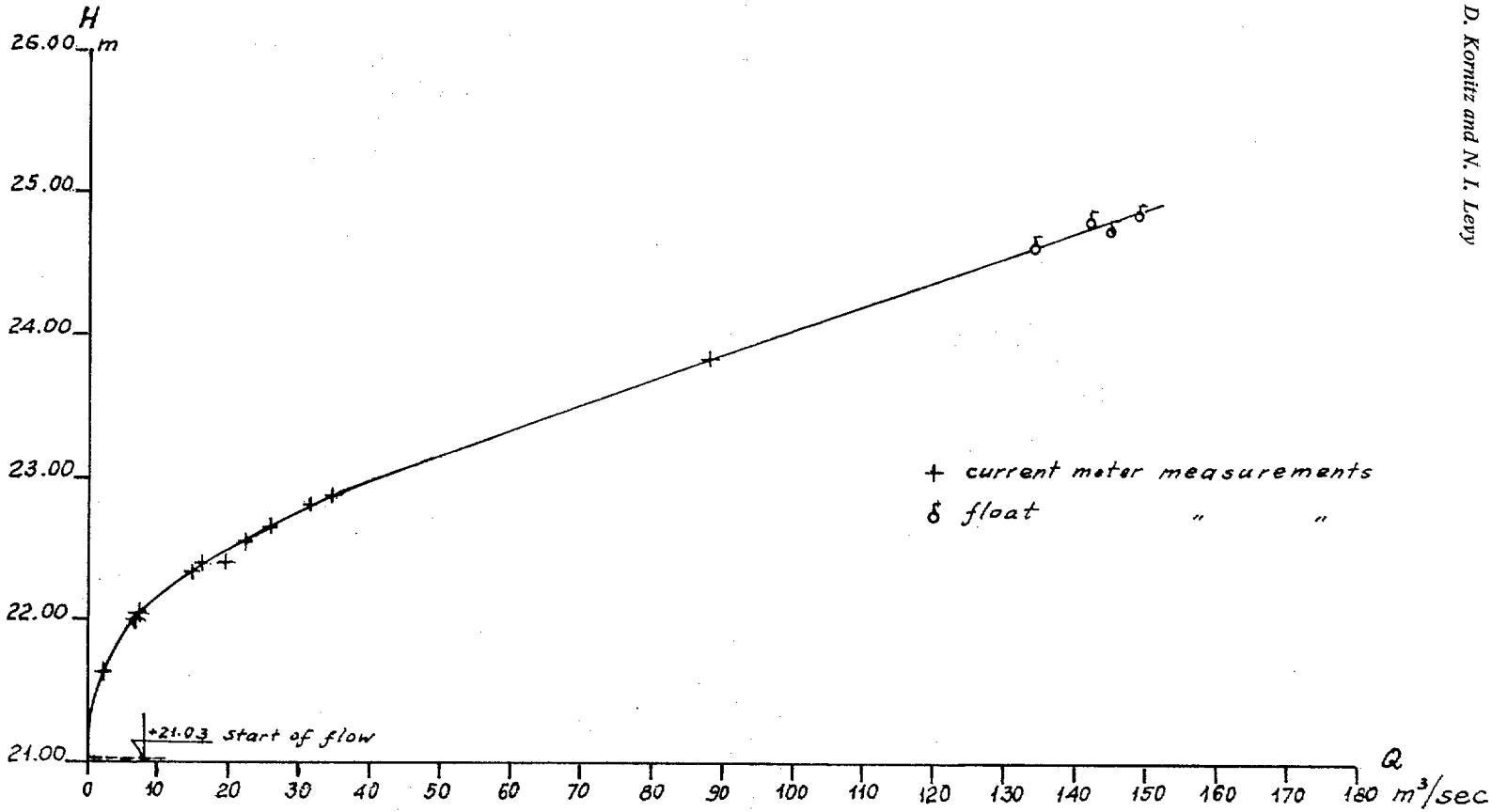


Figure 4 – *Nahal Ayalon at Beit Dagan-Yehud Road. Stage-discharge curve*

A_2 is a point at the intersection of this 45°-line and the vertical drawn from point A on the stage-discharge curve:

$A_1A_2 = OA_1 = Q_1$, the discharge corresponding to water-level H_1. By the same procedure $B_1B_2 = OB_1 = Q_2$, the discharge corresponding to water-level H_2 (point B on the stage-discharge curve).

The technique of drawing hydrographs from limnigraphs with the aid of the stage-discharge curve is, therefore, as follows:

(a) Trace the limnigraph on to transparent paper (H-curve). Draw base-line at height H_0 (level corresponding to zero discharge).
(b) Draw stage-discharge curve with same scale for H as that of the limnigraph. Draw line at 45° through the head of co-ordinates axis.
(c) Fix stage-discharge curve on the drawing-board.
(d) Attach the lower end of the traced H-curve to a ruler. Slide it along a second ruler fixed on the drawing-board in such a manner that the H_0 base line slides along the Q abscissa (line O-A_1-B_1-Q of the stage-discharge curve in Fig. 6).
(e) Bring point on H-curve to be translated into Q to rest on the stage-discharge curve (for instance point A) and draw vertically the corresponding point on 45° line (A_2). This gives one point of the hydrograph. Sliding the H-curve left or right, as many points as necessary to draw the whole hydrograph are plotted in this manner.

If it is necessary to change the scale of Q in the hydrograph (see example in Fig. 6), it can be done graphically by changing the inclination of the O-A_2-B_2 line. If, for instance, the line is drawn from O to a point lying half-way between A_1-A_2, then all the points along this inclined line will give a hydrograph drawn to the half-scale Q of the original stage-discharge curve. In the same manner, the Q-scale may be doubled, tripled, or drawn to any scale desired. This procedure enables one to draw the hydrograph very accurately and in a comparatively short time.

7. Water-level records (limnigraphs)

The measuring site is visited occasionally by the technical staff. Upon those visits the exact time is marked on the graph and the water-level at that time is observed. These marks and observations enable us to compute the true time and water-level and to check the performance of the recording instrument. The following variations are encountered.

1. Figure 7 shows a strip-chart with a flood occurring in the eight days between two visits. At a scale of 48 mm per day, the distance will be 384 mm between the time-marks. At remote sites of the Negev, visits are made once in two months or more. Then the distance between the time-marks may be more than 3 m. The time scale is computed, dividing the distance by the true time between the two time-marks.
2. Figure 8 shows a limnigraph of a recorder whose clock stopped after the visit of 8 February 1968. The water level continued to fall and our next visit on 9 February 1968 at 8°° we found a straight line drawn by the pen of the recorder from the time that the clock stopped till our visit. The time-mark made at the end of that vertical on 9 February 1968 at 8 hr, and observations of water-level made by our staff later on, made it possible to draw the whole limnigraph for this flood.
3. Figure 9 shows a limnigraph of a drum-type water-level recorder. The drum makes one revolution per week, and after each revolution the writing stylus drops 5 mm so that the base-line of the chart is drawn as stepped parallel lines spaced 5 mm apart for each revolution. In this manner it is possible to calculate the exact time of each step on the chart, and from it, the time of beginning and end of the flow (or any point on the chart).

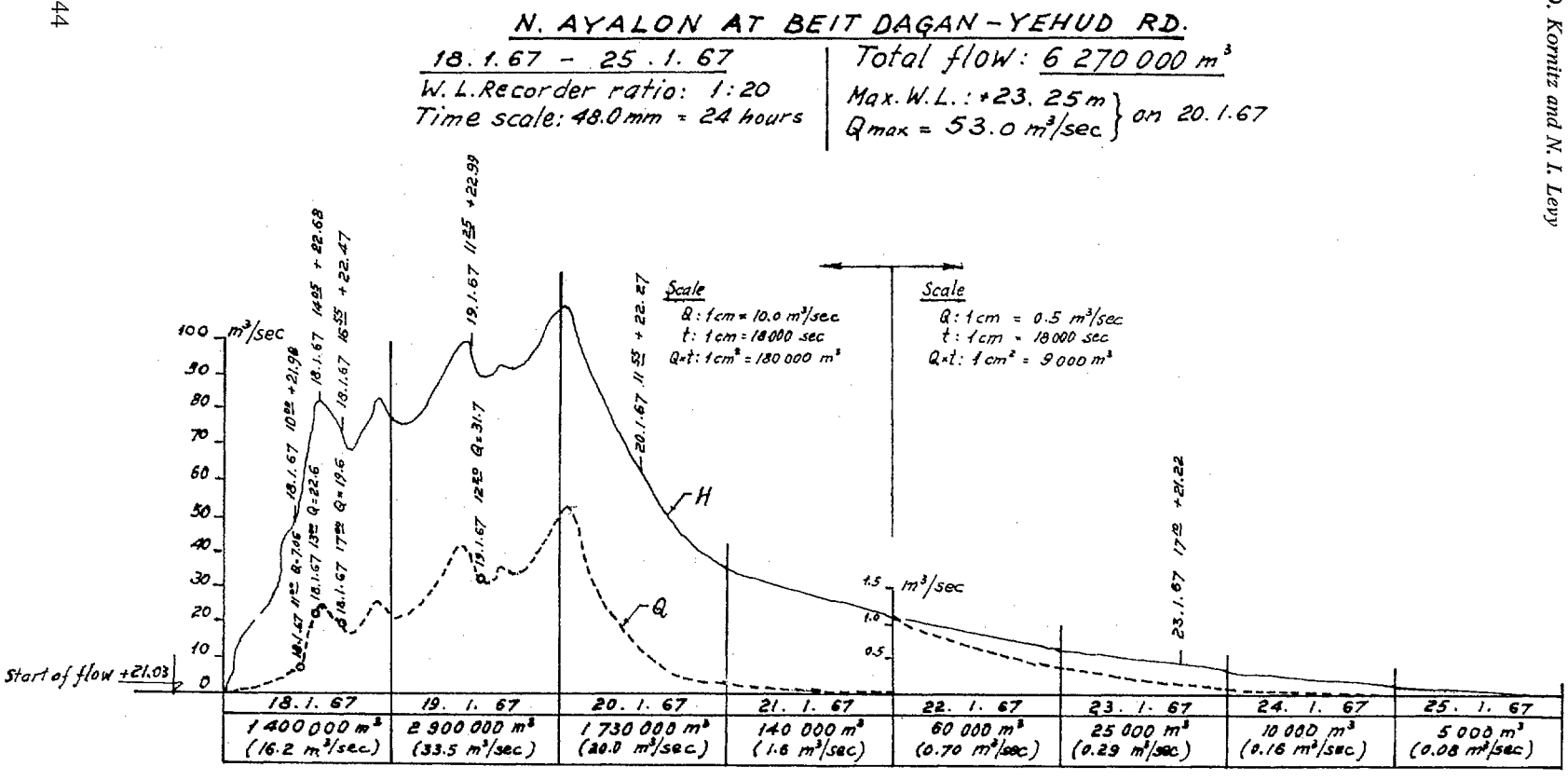


Figure 5 — *Hydrograph of the Nahal Ayalon*

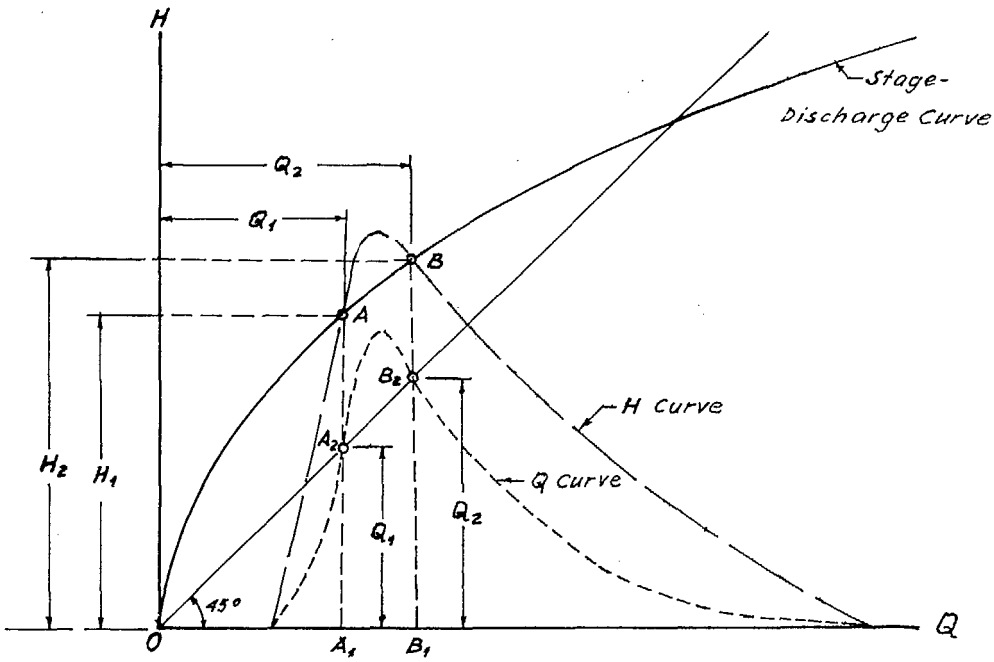


Figure 6 – *Rapid evaluation of flow from a limnigraph*

4. *Reversed chart of a drum-type recorder* Figure 10 shows schematically the performance of a drum-type water-level recorder. Float 1 moves upwards with rising water-level in the stilling well (in the river), and transfers its movement through the wire to wheel 2 which turns in the direction of the arrow (in our case, clockwise). Wheel 2 turns on its axis 3 and causes the scale-transmission wheel 4 to turn in the same direction. Transmission wire 5 is unlooped from wheel 4 and its released length going through wheel 6 allows counter-weight 7 to slide down on beam 8. The stylus 9 then moves downwards on the diagram fixed on drum 10 (drum 10 is being turned by clock 11). This downward movement of the writing stylus is possible as long as a length of wire 5 is looped around wheel 4.

However, at the start, a sufficient length of wire is looped on to allow the stylus to reach the lowest point on the diagram paper. The moment that this wire is entirely released from wheel 4, and the water table continues to rise in the river, then the end of wire 5 fixed to the wheel 4 moves from the left to the right side tangent to wheel 4, and it is then looped on wheel 4 again, but this time at the opposite direction than when released from wheel 4. This movement of the wire shortens its released length and causes the writing elements (7 and 9) to slide upwards on beam 8.

The resulting diagram is shown in Figure 11. At the points of reversal there is a small error E on record which prevails for the whole length of the record between the reversal points. As long as wire 5 is looped around wheel 4, its released length for each revolution is $\pi . d$ (d being the diameter of wheel 4). When wire 5 is entirely released, then for approximately half a revolution of the wheel, its end moves up and down again a length equal approximately to the radius of wheel 4.

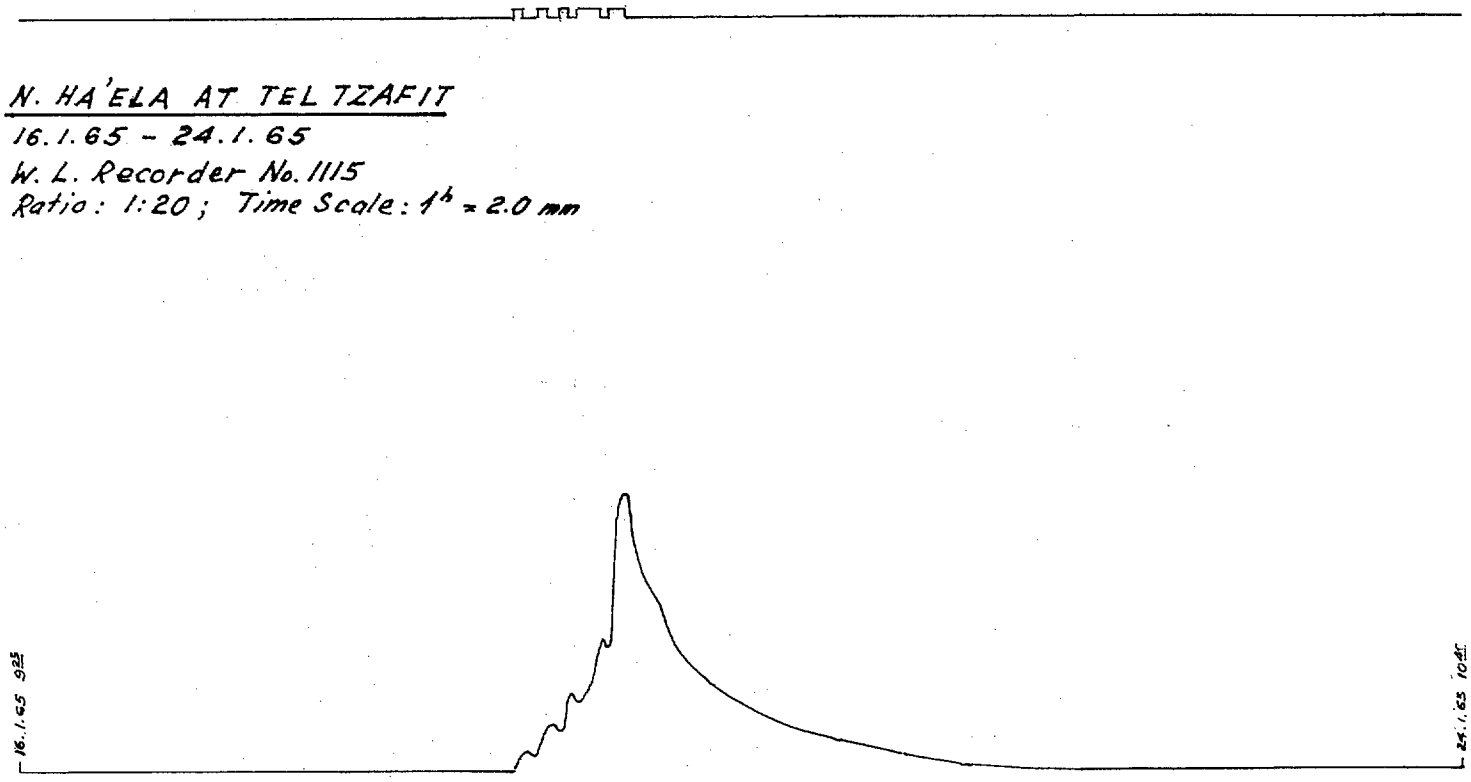


Figure 7 – *Limnigraph showing flood occurring between two visits by staff*

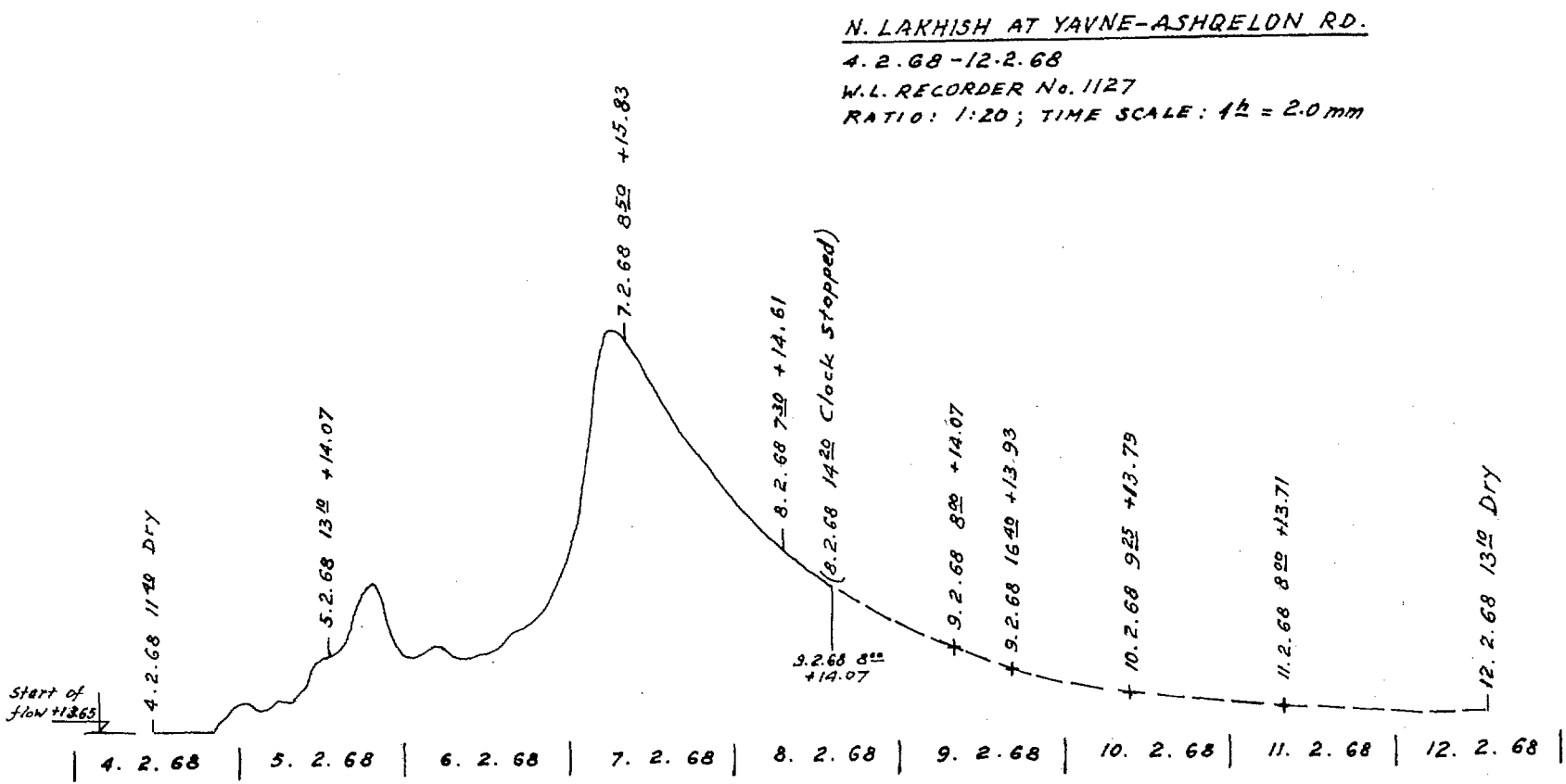


Figure 8 – *Limnigraph from recorder in which the clock stopped. The dotted line represents readings taken by staff latter on*

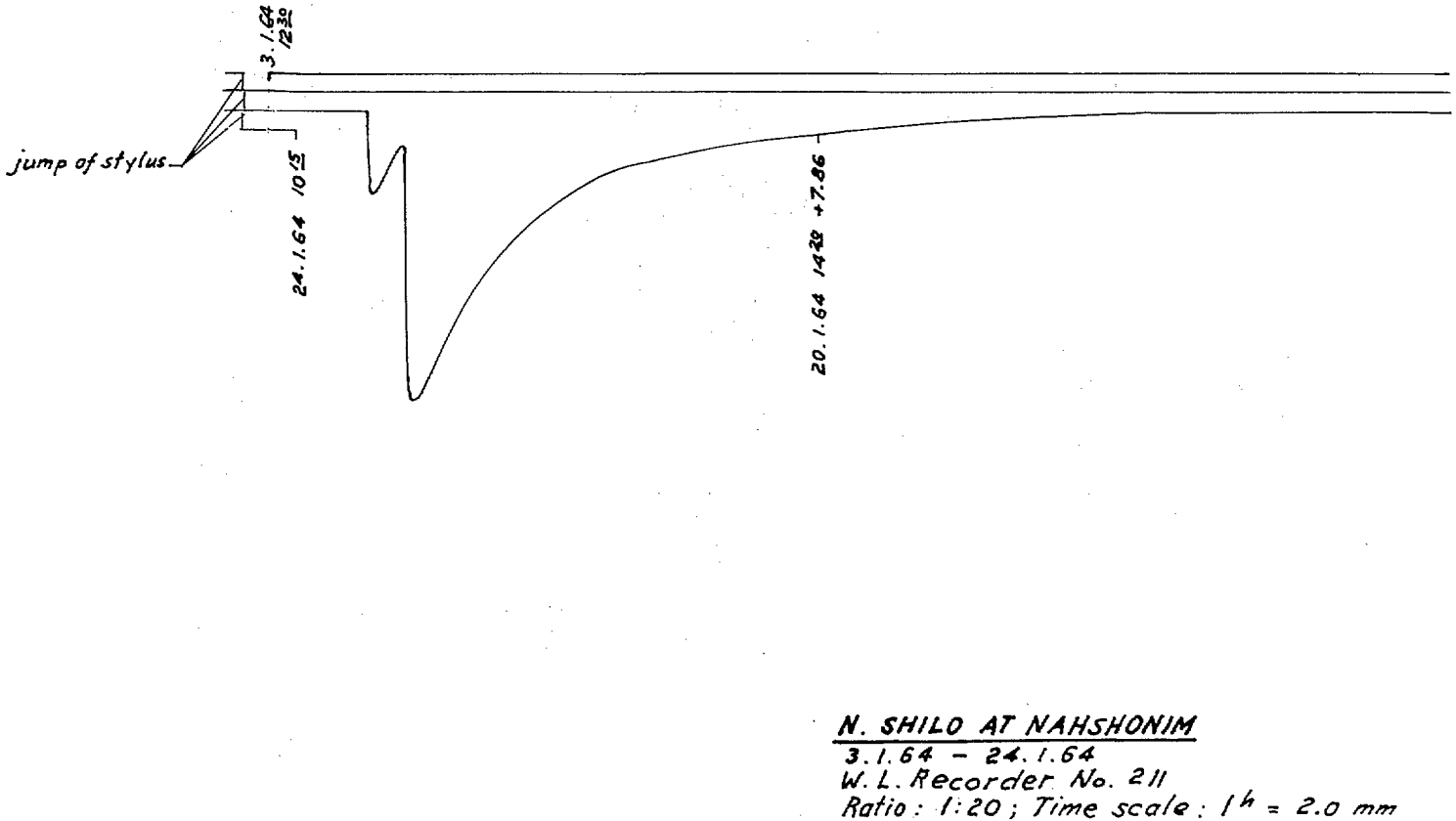


Figure 9 – *Limnigraph of a drum-type water-level recorder*

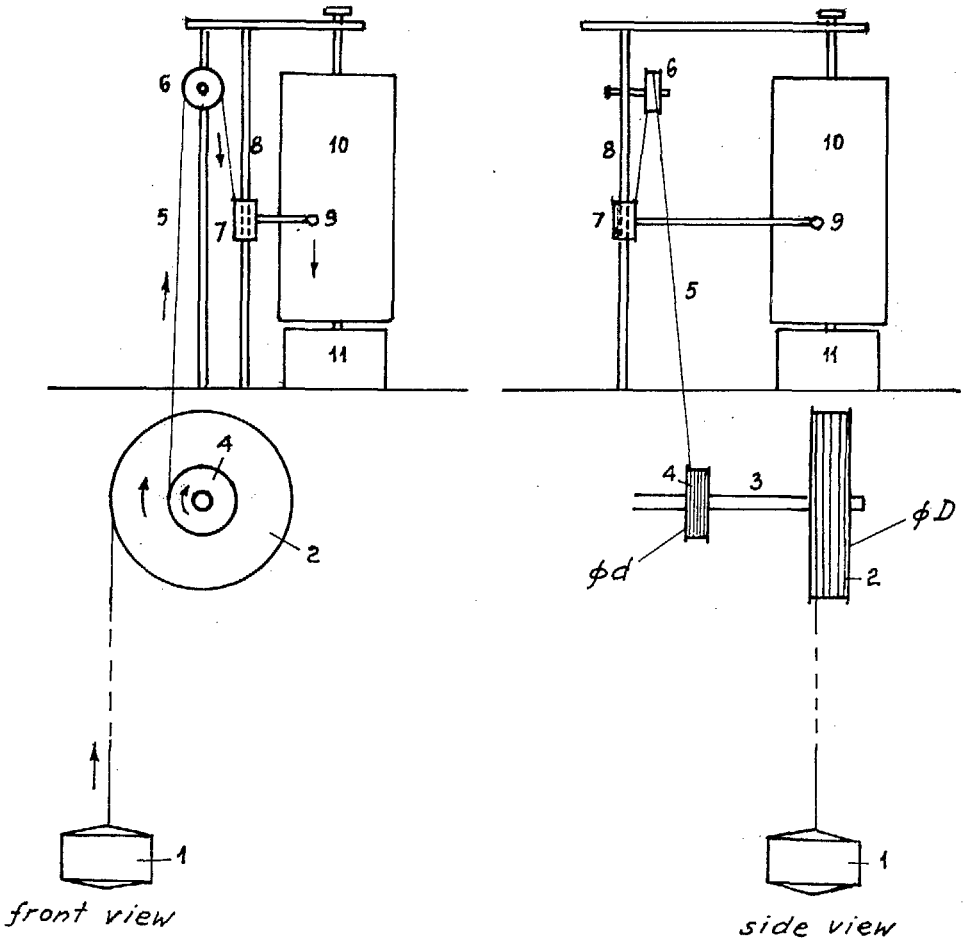


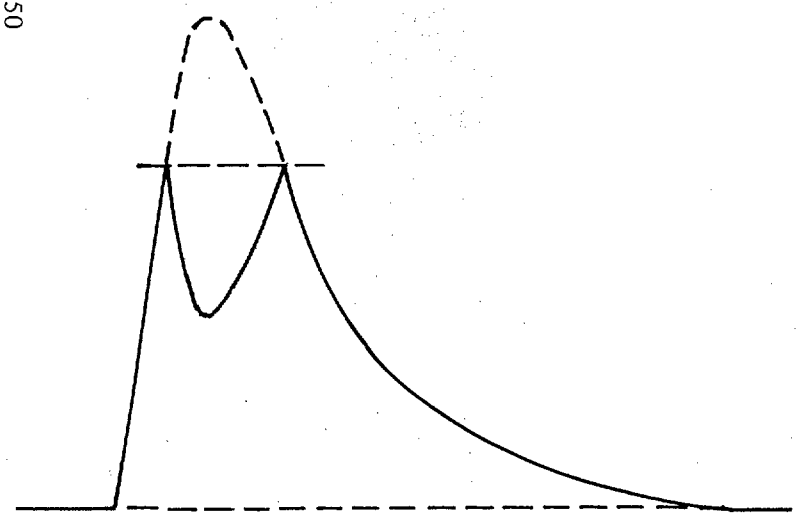
Figure 10 – *Schematic representation of a drum-type water-level recorder*

It means that the stylus 9 draws a height equal to $d/2$ instead of $\pi \cdot d/4$ for one-quarter of a revolution, the difference being $(\pi \cdot d/4) - d/2$ (which is $\simeq 3$ mm for a transmission wheel 10 mm in diameter).

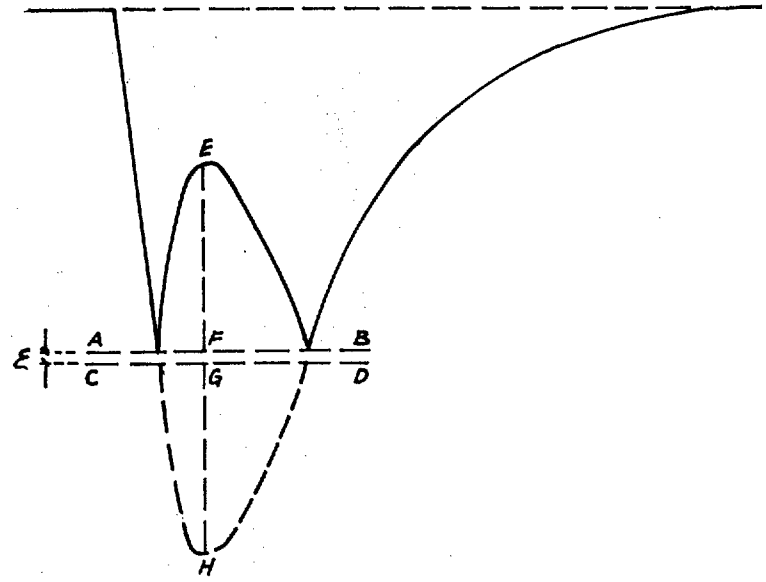
5. *Reversed diagram of a strip-chart recorder.* In case of the strip-chart recorder (Fig. 11), there may be an error of the record at the reversal point. This error depends on the construction of the instrument. In the Israeli instruments the cam and follower of the writing elements is a work of high precision and there is practically no error at the reversal points.

8. *Hydrograph of spring flow*

Base flow after flood, decay of interflow, and spring flow, are computed directly from current meter measurements of discharge made, as a rule, once a month. Fig. 12 shows the hydrograph of the spring Ein Naftali, drawn and computed for the hydrological year 1964/65. The discharge measurements are plotted to scale on their appropriate dates (marked as small circles) and straight lines connect measured



For strip-chart W.L. Recorder

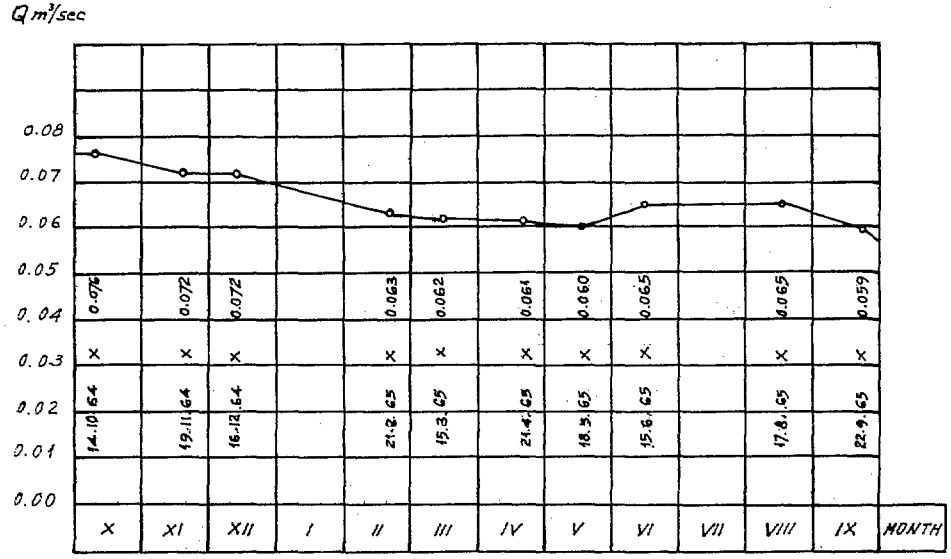


For drum-type W.L. Recorder

Figure 11 – *Reversed diagrams for strip-chart and drum-type water-level recorders*

STATION: EIN NAFTALI

	EAST	NORTH
APPROX. CO-ORDINATES (KM):	197.52	208.22



DATE	Q m³/sec	
25.11.48	0.094	KNOWN MAXIMUM
19.5.60	0.031	KNOWN MINIMUM
63/64 - 64/65	0.068	AVERAGE

CHEMICAL DATA

DATE	CL mg/l	ELECTRICAL CONDUCTIVITY M/mhos/cm	TOTAL HARDNESS FRENCH DEGR.	pH
9.11.64	290	1585		
5.4.65	300	1534	50	

TOTAL	201	188	191	181	154	166	158	162	166	174	172	155	$10^3 m^3$
AVERAGE	0.075	0.073	0.072	0.068	0.064	0.062	0.061	0.061	0.064	0.065	0.064	0.060	m^3/sec

METHOD OF DISCHARGE MEASUREMENT

BY WEIR ⌊
BY CURRENT METER ×
BY FLOATS ○
VOLUMETRIC V

Figure 12 – *Hydrograph of the spring Ein Naftali for the hydrological year 1964/65*

points. By interpolation, the discharge at the beginning and end of each month is computed. From these figures monthly and early flow and averages are computed and tabulated as shown in Figure 12.

9. Summary

With the development of Israel, new tasks have been added to the Hydrological Service. As in other countries in the world there are difficulties in recruiting additional staff. It was decided to free a part of the existing manpower from the tedious and time-consuming work required for all these calculations described in previous paragraphs and use available staff to better advantage in the hydrological research work.

PART II. AUTOMATIC PROCESSING OF HYDROMETRIC DATA AT THE HYDROLOGICAL SERVICE OF ISRAEL

1. Common approaches

It is an accepted prerequisite that, for automatic processing of hydrometric data, the hydrometric observations should be in a digital form adequate for computer input. As yet, the most commonly used recording instruments in hydrometric stations all over the world are of the analogue type, producing a graphic record of the observations. To process these data (records) with the help of a digital computer, it is a common practice to use a graph-reading instrument of one type or another, e.g. a pencil follower, an XY digitizer. However, the use of such an instrument necessitates the registration and storage, whether on punched cards, tapes, or magnetic tape, of a large quantity of information, namely the two co-ordinates of all sampled points. This information must then be transferred to a digital computer for processing with all the familiar problems associated with information transfer.

Another known solution which can overcome part of the problems inherent in the above method is the use of digital recorders, thus providing the source information in a digital form. The use of digital recorders, no doubt, enables the achievement of a higher level of automation in the processing of the hydrometric data. This, however, is at the cost of losing the advantageous graphic representation, at least at the early stages. Furthermore, it necessitates the replacement of all existing analogue recorders by digital ones. Beside the expenses involved, this also has other consequences on organization and maintenance, etc. It should also be mentioned that, although the information obtained from the digital recorders is in a digital form, in most cases, whether punched tape or magnetic tape is used, it is not computer compatible. Thus, translation and special translation equipment are required.

2. A new approach

The two methods described above are applied in various countries. Although they represent modern techniques, they are relatively time-consuming, considering the time elapsed from the moment the field records have reached the office till final results of the processing are obtained. This is especially the case where use is made of an external or a service bureau computer.

Both solutions are actually based on the aforementioned prerequisite, namely that the observations should be in a digital form. In our opinion, this is not necessarily so. There is no reason why we should not use the analogue record, widely available as it

is, and input it directly to a digital computer without any separate, intermediate conversions or translation.

In this article a description is given of a different approach to the problem of automatic processing of hydrometric data, namely the use of a special purpose system, the Levy Graphicomp System (LGS) developed by N. I. Levy. It is a low cost, simple but powerful system, accommodating all required facilities to enable a complete and immediate analysis and processing of hydrometric observations using directly the analogue graphic records. LGS thus enables the maintenance of the already existing analogue recorders, and provide simple and straightforward means for complete processing of these records in the office within minutes.

3. The Levy Graphicomp System[1]

The LGS system, shown in Figure 13, is an integrated software-hardware system accommodating graphic input/output facilities as well as processing means. LGS enables analysis and processing of graphs and curves on a real time basis, obviously in respect to the time the graphs are read. There are two standard versions of LGS: basic and extended. The basic LGS system consists of following major units (Fig.14-blocks drawn in full lines).

1. Curve follower (A precision XY digitizer);
2. Central processor;
3. Functional control keyboard;
4. Display panel;
5. Teletype (TBM).

The basic LGS is expandable and may include, in addition, the following (Fig. 14-blocks drawn in dashed line):

6. Additional core memory;
7. Secondary storage (magnetic tape and/or drum);
8. *XY* plotter;
9. High speed paper tape input output.

The curve follower is a precision *XY* digitizer of the mechanical type, having a stylus movable by free hand in a Cartesian co-ordinate axis system. The stylus movements are practically frictionless, thus enabling the smooth following of curves of any shape. The standard curve follower is a tiltable flatbed table having workable area of 60 cm x 80 cm ($\simeq$ 23 in. x 31 in.) and 0.05 mm (0.002 in.) resolution.

The central processor is a mini-computer with associated electronics. It has a 4K words core memory. Cycle time is 2 μsec.

The functional control keyboard is used to control the operation of the system. It contains a set of twelve sense switches, twelve functional push buttons, tumble switches, and a nine-state display selector controlling the display panel.

The display panel consists of two sets of six Nixie tubes each for numerical display, and in addition, twelve indicator lamps, the front panel of which is exchangeable.

For digital input and output, as well as for clear text output, a Teletype model TBM is used. As no great volumes of output are involved in normal operations, the type and speed of the output unit are of secondary importance for most applications. However, if required, high speed input/output units can be provided.

4. Automatic processing of hydrometric data with LGS Model H

LGS Model H is a version of LGS especially adapted for hydrometric data analysis.

1. Patent and trade mark pending.

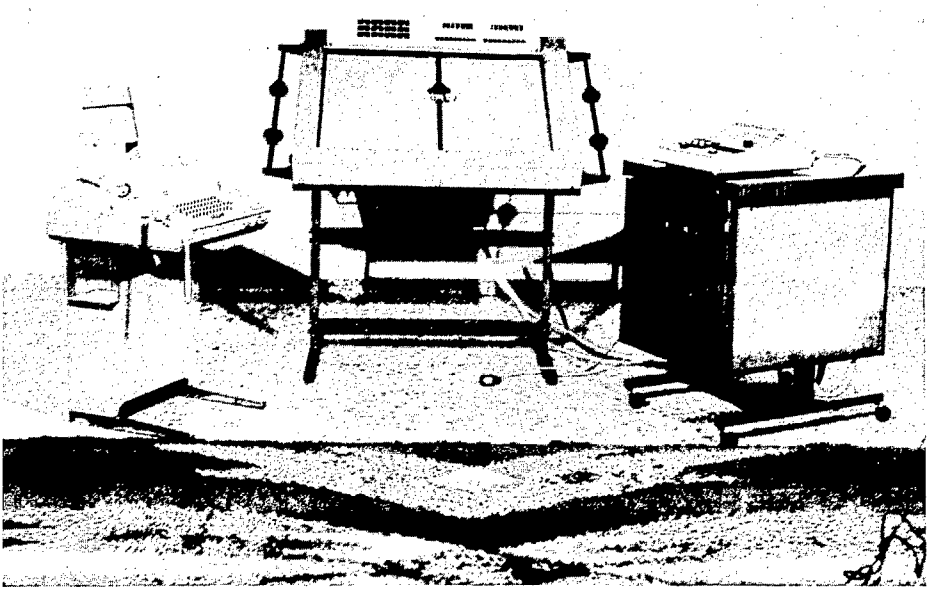


Figure 13 – *Levy Graphicomp System (LGS)*

Such a system has recently been installed in the Hydrological Service of Israel. LGS Model H is applied for the processing of various subjects. Here, however, we shall limit ourselves to the processing of limnigraphs only.

The functions of a LGS-H have been defined in close co-operation with the Hydrological Service of Israel. LGS-H was so designed as to avoid as much as possible human intervention within the processes, and to simplify its operation for operator convenience. All analogue graphic records, as obtained from the various analogue recorders applied in the Hydrological Service (see description in Part I), can be analysed and processed directly. By "directly" we mean that the *original* graphic records, whether of the drum type or chart type, can be analysed without any intermediate translation or conversion.

The analysis of a limnigraph is quite simple; the operator sets the original limnigraph arbitrarily on the curve follower and follows it in free hand with the help of the stylus, after inserting the stage discharge curve and reference points. LGS performs all processing and analysis of the limnigraph in real time, i.e. in respect to the time the limnigraph is followed.

The operator controls the operation of LGS and selects various functions, as required, through the control keyboard. LGS has two modes of operation: Automatic and Manual which are switch dependable. In the automatic mode the limnigraph, when followed, is continuously (minimum increment 0.05 mm) and automatically registered and analysed. In the manual mode, however, registration is under a push button or a foot switch control, thus selective registration is possible. The mode of operation can be changed at any time within the processing stage. This enables us to achieve a high level of accuracy and efficiency, since straight line segments need not be followed, only their end points need be registered.

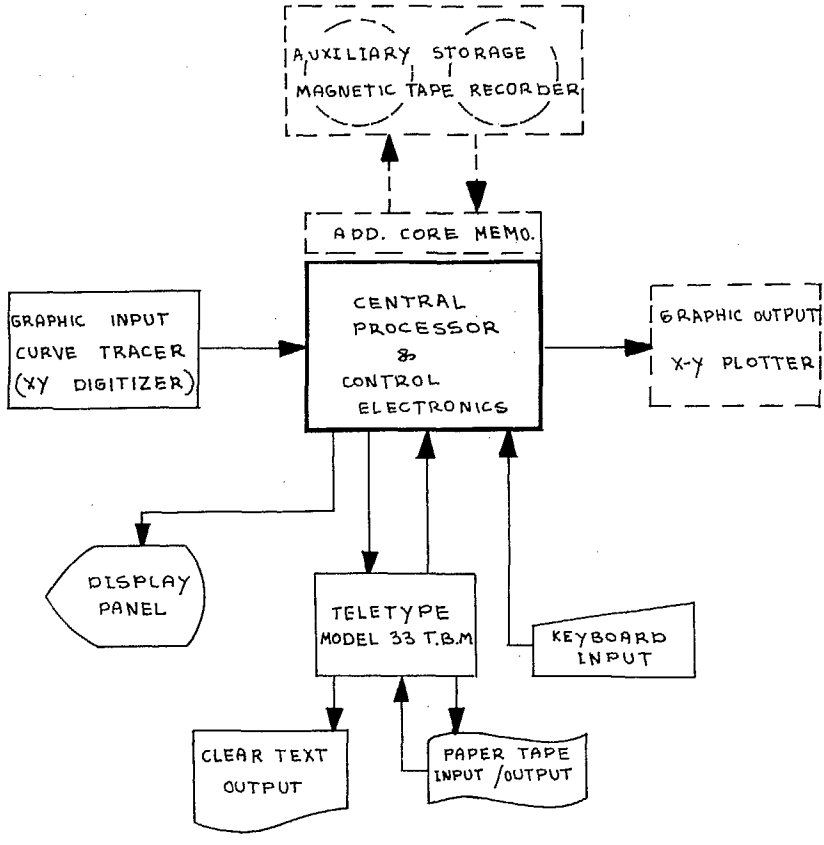


Figure 14 – *Block diagram of the LGS*

Other functions are under operator control, such as transformation or inversions. This makes it possible to perform complete analysis on the original records, whether of the drum-type or strip-chart, with all their specific recording problems as described in the first part of this paper.

Furthermore, a transformation is applied to the limnigraph, based on "control points". These control points are obtained from human observations taken while the station and recorder are being visited for checking. With this transformation LGS corrects for systematic errors inherent in the recording. In addition, it obviates the need for a precise orientation of the limnigraph on the curve follower.

While the limnigraph is being followed, LGS analyses it continuously in real time. Only final results, e.g. daily discharge, are output. However, the display system enables the operator, for his convenience and for checking purposes, to display other values within the process.

5. Summary

LGS-H offers for the first time an in-office complete system for processing of hydrometric data using directly the original analogue records. It obviates the need for conversion or translation of any kind and eliminates transfer of large quantities of

information. It enables a complete analysis in one system, by one operator, and reduces the time required for analysis from days to minutes. Furthermore, results can be edited and printed in a form adequate for final publication in the hydrological bulletin.

LGS is an integrated software hardware system. It is programmable and thus general purpose within its processor capacity. LGS can be used for the processing and analysis of various data of different subjects in hydrology and meterorology as well as in other fields. With the addition of a plotter the range of applications is still wider. Analysed results can be presented also in a graphic form in addition to the digital representation.

ACKNOWLEDGEMENT

The authors wish to express their gratitude to Mr. M. Jacobs, Director of the Hydrological Service of Israel, and to Dr. S. Shmurak, for their help and encouragement in developing LGS-H.

Experiences with a fully automatic curve scanner

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Federal Institute for Hydrology, Koblenz,
Federal Republic of Germany

Abstract. The evaluation of gauge records used to be cumbersome and time-consuming. In the course of the rationalization and automation of hydrometry, the Federal Institute of Hydrology has been searching for ways to simplify and accelerate this work since the early sixties. A fully automatic unit for curve scanning and evaluation was designed in co-operation with industry as early as 1964, and after three years of development it was tried out late in 1967. The paper gives a brief description of the unit and a report on two years of practical experience gained in its use in the evaluation of stage hydrographs. A comparison with the manual evaluation previously practised reveals the unit's pronounced superiority in performance and economy.

In conclusion, a report is given on another type of work accomplished with the aid of the unit, i.e., the scanning and evaluation of cross-sections as a prerequisite for hydraulic calculations.

EXPERIENCES REALISEES AVEC UN DECHIFFREUR DE COURBES ENTIEREMENT AUTOMATIQUE

Résumé. Dans le passé, le dépouillement des enregistrements de limnigraphes était très compliqué et demandait beaucoup de temps.

En procédant à la rationalisation et à l'automatisation de ces activittés en matière d'hydrométrie, l'Institut fédéral d'hydrologie, au début des années soixante, a étudié les possibilités de faciliter et d'accélérer cette opération. Après avoir conçu et développé un appareil semi-automatique pour le dépouillement des courbes de niveau, l'institut mit au point, dès 1964, en collaboration avec l'industrie, un dispositif entièrement automatique de balayage et de calcul, qui, après trois années d'essais et de perfectionnements, s'est révélé entièrement satisfaisant. On donne une description du dispositif et l'on relate les résultats obtenus au cours de deux années d'utilisation.

Une comparaison avec le dépouillement manuel d'autrefois montre à l'évidence la grande supériorité de ce dispositif, tant du point de vue performance que du point de vue économique.

Finalement, l'auteur donne des précisions sur d'autres travaux effectués entre-temps à l'aide de ce dispositif : mesure et tracé des profils des sections transversales, opérations nécessaires aux calculs hydrauliques.

EXPERIENCIAS CON UN TRAZADOR AUTOMÁTICO DE CURVAS

Resumen. En el pasado, la evaluación de registros de limnígrafo era muy complicada y exigía mucho tiempo.

En el curso de la racionalización y de la automatización en el campo de la hidrometría, el Instituto Federal de Hidrología ha estudiado, al comienzo de la séptima década del siglo XX, posibilidades de facilitar y acelerar dicha operación.

Después de haber desarrollado un aparato semiautomático para la evaluación de las curvas de nivel se concibió ya en 1964, en colaboración con la industria, un dispositivo totalmente automático de trazado de curvas y evaluación, que al cabo de tres años de desarrollo ha podido ser sometido a prueba, es decir, a finales de 1970. En la comunicación figura una descripción del dispositivo y un informe relativo a las experiencias prácticas adquiridas durante dos años.

Si se hace una comparación con la evaluación manual de antaño, se pone en evidencia la gran superioridad de este dispositivo, desde el punto de vista de economía y de rendimiento.

Finalmente, el autor explica otros trabajos realizados, entre tanto, con la ayuda del dispositivo, como el trazado y evaluación de perfiles transversales, dato previo para los cálculos hidráulicos.

ОПЫТ ПОЛЬЗОВАНИЯ ПОЛНОСТЬЮ АВТОМАТИЗИРОВАННЫМ ПРИБОРОМ ДЛЯ ЗАПИСИ КРИВЫХ

Резюме. Обработка уровенных записей-трудоемкая опереция. В ходе рационализации и автоматизации гидрометрических работ Федеральный институт гидрологии искал пути упрощения и ускорения этой работы с начала 60-х годов. Следуя развитию гидрографии, в 1964 г. был сконструирован в кооперации с промышленностью полностью автоматизированный агрегат для вычерчивания кривых и их обработки. После 3-х летней разработки он был впервые испытан в конце 1967 г.

В докладе дается краткое описание агрегата и отчет о двухгодичной его эксплуатации, направленной на получение графиков колебания уровней. Сравнение его с ручной обработкой, применявшейся ранее, обнаруживает значительное превосходство этого агрегата в работе и экономию.

В заключении приведены данные о работе другого типа, выполненной с помощью этого агрегата, а именно, вычерчивание и оценка поперечных профилей, как условие, необходимое для выполнения гидравлических расчетов.

As a result of post-war economic recovery in the Federal Republic of Germany, and of the rationalization and automation in all fields of human activity which accompanied it, recorders are now employed in continuously growing numbers to supervise and control machine data and production cycles. The work involved in the manual evaluation of curves and hydrographs grew to such an extent that it could no longer be done by hand. To overcome this difficulty, a fully automatic scanner of recording tape of up to 120 mm width was brought out in the early sixties. The scanning element was a vidicon or camera tube. Unfortunately, the instrument could not be used in hydrology because its scanning width was insufficient. Yet in hydrology a steadily growing amount of work was required for the evaluation of stage hydrographs; at the same time shortage of personnel become a problem.

Hence the Federal Institute of Hydrology began to search for ways and means to mechanize and accelerate this work. The first result of these efforts was a semi-automatic instrument for scanning curves and converting stage hydrographs into discharge curves. Developed at the suggestion and under the direction of Dr. Eschweiler, the instrument became known as the hydrograph converter and helped to save a considerable amount of personnel time.

Even so, the instrument was not a final solution. What was needed was a fully automatic arrangement, which would exploit improvements in technique of data processing. In 1964, a fully automatic installation was planned in close co-operation with industry and during the three following years it was developed to technical maturity. Essentially, the installation consists of three parts:

1. Diagram scanner;
2. Computer with electronic memory;
3. Output unit.

The chart scanner employs a cathode ray tube the electron beam of which is sharply focused and periodically deflected at a frequency of 50 Hz (standard frequency). The electron beam hitting the chart produces a fine light spot of 0.3 mm diameter which is moved 50 times per sec, in the ordinate direction, over the full width of the chart while the latter is moved at a uniform speed in the abscissa direction, with the result that the light spot actually zigzags across the chart. The distance between two successive scannings is between 0.1 mm and 0.8 mm, depending on the paper speed,

which is adjustable. The light spot is reflected by the relatively light ground of the chart, and the reflected stray light is directed into two photomultipliers. When the light spot cuts the curve, the intensity of the reflected stray light decreases sharply so that a signal is produced in the photomultiplier. While the light spot travels across the full recording width, an impulse generator supplies 250 or, alternatively, 500 impulses. As soon as the light spot crosses the zero line, the impulses are fed into an electronic counter, to be stopped again by the signal in the photomultiplier when the light spot hits the curve. Consequently the number of impulses fed into the counter between the zero line and the intersection of the electron beam with the curve is proportional to the amplitude of the trace. Actually, the beam, when cutting the curve, alternatively scans the latter's lower and upper edge so that it is exactly the centre line of the path that is registered. This is important, especially with wide records such as result from poorly dampened wave motion.

The instrument has been designed for scanning recording charts of up to 250 mm width. The co-ordinate print must be of a colour that will not disturb the scanning process, and there must always be only one curve, and no additional entries, ink spots or stains underneath the curve.

Evaluation in the component computer at the Federal Institute of Hydrology involves calculation of the daily and monthly means of water stage and selection of the lowest daily mean and the highest absolute value of the month, with indication of the days on which they occurred. Four-hour means can be calculated instead of daily means if the daily water level is subject to heavy fluctuations and the mean discharge cannot be calculated correctly from the daily mean.

Output of the values is effected by means of a line printer to which a five-channel tape punch and tape reader are connected. For the daily means and extreme values for a month, the punched paper tape has a length of about 35 cm.

For various statistical purposes, in particular the compilation of hydrological yearbooks, the stage values thus determined are insufficient. It would not have been difficult to expand the computer hardware to permit the determination of all appropriate discharge data including frequencies and duration values. Instead, the Federal Institute of Hydrology assigned this task to an independent computer. Thus the data can be printed either according to the yearbook pattern or in any other form. The Institute has developed this programme in FORTRAN, for further processing of values of water stage. The computer with associated plotter will also be able to present the measurements graphically. This might be of interest in a number of cases.

Such a scanning and evaluating installation has now been in operation at the Federal Institute of Hydrology for about two years. It was procured to evaluate the records of the first- and second- order inland gauges of the Federal Waterways Administration. Even today, the records of most of these gauges are still registered on charts and must, therefore, be transferred to a suitable chart before scanning. To transfer, for example, the record of one gauge month, a trained assistant will, on average, need 15 min. That is more than is required for the subsequent scanning and evaluation. The aim is, of course, to have water levels recorded directly on paper tape rather than on weekly charts. When purchasing new equipment, the Federal Waterways Administration and the regional authorities now decide in favour of punched tape recorders because these eliminate the need for the weekly change of chart and all the error sources that go with it.

After the scanner has been set for the desired paper width and the chart has been inserted, the apparatus works fully automatically at speeds of 5, 10, 20 or 40

mm/sec. This means that the actual evaluation of a one-month strip chart takes only about 90 sec. Simultaneously the results are printed and recorded on punched tape. Discharge values and other statistical data are determined by a computer in a few seconds per gauge month, the actual time required depending on computing and output speed. Even though most gauge records have still to be transferred on to chart for evaluation, time savings, as compared with manual, evaluation are considerable. At a transfer rate of 15 min for each gauge month, after only a short period of training, an assistant can handle 30 gauge months daily. The further time required for the scanning and evaluation of the hydrographs and for the subsequent processing in the computer is almost negligible. On average, one person can thus transfer and scan 20-25 gauge months per day. This compares very favourably with the previous manual evaluation system, where a fully trained person needed about one day for every gauge month. In other words, mechanical evaluation requires only one-twentieth of the time needed for manual evaluation. Another important advantage of automatic scanning and evaluation is that error sources and subjective influences are eliminated as far as possible. Before the automatic evaluation takes place, the records are, of course, checked visually and corrected where necessary, as with manual evaluation.

For water stage recordings, an accuracy of ± 1 cm is required. The same accuracy is applied to the transfer and scanning of gauge records. To ensure this high accuracy, the scanner is carefully adjusted and set each time it is to be used, and this is done with the aid of two calibration verticals and potentiometers, the proper setting of which can be checked with the help of control lamps or of an oscilloscope. It is advisable to repeat this check from time to time because the gradual warming-up of the installation during operation, or its cooling-down, may lead to a drift in calibration. In addition, it was found that the installation should be turned on at least half an hour before the start of the scanning operation and that it should not be turned off if evaluation is to be interrupted only for a brief interval. To avoid overheating at high ambient air temperatures in summer, the evaluation room has been air-conditioned.

During the first year of operation, about 1200 monthly gauge records were scanned and evaluated. With manual evaluation, one person would have required about six years to do the same job. Thus the entire installation, which cost about DM.90,000, was fully amortized during the first year.

As these gauge record evaluations did not, by far, exhaust the installation's capacity, additional tasks were assigned to it, for example, the scanning and evaluation of cross-sections. The ordinate values are fed into an electronic counter, and the areas to be planimetered are defined by means of so-called marginal marks, i.e., black points of about 0.5 mm diameter, below the zero line. The installation stops the scanning process automatically when the scanning beam reaches the point mark. In this way, any part of the cross-section can be scanned and planimetered. The automatic scanning of cross profiles is of interest because experience shows that, due to human fatigue, manual planimetering cannot be effected at the required high level of accuracy over any prolonged period of time.

In addition to evaluation of gauge records and cross-sections the installation also scans and evaluates water quality recording tape. This task will grow considerably in scope and importance during the coming years.

DISCUSSION

Krapp: What type of computer have you built into this device? Is it a digital computer of the invariable-programme or the variable stored programme type?

Schäfer: The permanently built-in computer is of the invariable-programme type.

Krapp: By way of certain manipulations you can also change these programme routines. You said it would be possible to modify this computer to enable it to include discharge calculations also. Is that possible with this unit?

Schäfer: Of course it is possible. However, it would not be advisable because the costs for a variable-programme computer would be very high. This permanently wired installation supplies all initial values required to subsequently have a variable-programme installation calculate all possible data and values.

Krapp: Only two manual operations are required, one of these being the transfer of paper strip information to tape, and the other the transfer of punched tape data to another computer.

Schäfer: Our installation delivers the daily means, the monthly mean, the monthly sum of water levels, and the two extreme monthly values with the date at which they occurred. For many statistical cases, this is sufficient. For other purposes including, for example, the year books, additional data, in particular discharge data, must be determined, and for this purpose a variable-programme computer is used.

Krapp: Can you indicate the proportional cost of scanner and evalution equipment? You mentioned a total price of DM.90,000. How much of this sum is for the scanner?

Schäfer: The scanner costs about DM.30,000.

Colenbrander: Can the original data be punched directly into tape at time intervals of 15 or 30 minutes?

Schäfer: The scanner scans the curve step by step. Our installation is so designed that it sums up the values of each day and then draws the mean. As a matter of principle, this installation, i.e. the scanner, can be connected to an output unit, for example, a Kienzle printer which delivers the individual ordinate values.

Colenbrander: This is not possible in a direct way, and in my opinion this is the difference as against the apparatus demonstrated by Mr. Kleinenbroich, which is of the variable-programme type.

Kleinenbroich: The programme covered only the determination of the boundary values of the water stages. The unit does not work like a variable-programmme computer as it did in my case. In my case, you can evaluate all programmes in connexion with water stage and discharge. This means that the unit is self-sufficient. On the one hand, it can work on-line where punched tape is fed in. On the other hand, you can also, and simultaneously, evaluate the water stages and the runoff

depth. You can then evaluate for mean values, i.e. for the main data, boundary values of maximum and minimum runoff depth, yield heights, and also for quarters of a year, half years and years in respect of water stage as well as of discharge.

Bielenski: Under conditions of normal gauge levels in the tidal region, we have in each period of about six hours a fluctuation upwards and downwards, and we need, therefore, a denser sequence of dots. What is the accuracy in time that can be reliably obtained with your instrument in evaluation, in particular as far as the combination of punched tape and printing is concerned? It is posssible to achieve in the five-channel system a good time control by using time marks, or is it necessary to use an eight-channel system?

Schäfer: Our installation is intended for the evaluation of inland gauges only. Scanning width is 200 mm normal and 250 mm maximum. With a typical tidal gauge you have a much larger scanning width. This is not possible at present. One must also remember that the computer has a permanently incorporated programme and is permanently wired so that, in its present form, it gives only the daily means for a certain length. In our case, with a paper advance of 2 mm/hr, this means a length of 48 mm. It is then always added, and the integrated as well as the mean values are drawn from it and printed.

And now to the question of time marking. In principle, it would be possible, but it would require changes in the present structure of the installation. We ourselves do not need these time designations.

Bielenski: Are you aware of the fact that, in the region of the tidal gauges, development has already progressed to the point where instruments of this sort have been designed? Or is it that no tests have been carried out so far?

Schäfer: Yes, scanning with semi-automatic instruments, such as mentioned by Mr. Kleinenbroich, would be possible because, here, ordinate values are determined and transferred at the beginning. In this case, too, the computer would have to be programmed differently.

Herschy: In the paper the author states that an accuracy of ± 1 cm is required for water stage recording. The interpreter translated this as ± 1 per cent, which of course is very different.

Schäfer: The percentage depends on the total height of the water level. At a 1 m stage, 1 cm means 1 per cent, at a stage of 10 m it means only 1 per thousand. Generally, however, the accuracy is actually reached over the entire stage, which means the fluctuation range.

Bliesener: Can any record sheet be fed in, or must the sheet be perforated along the edge? Is there, perhaps, an auxiliary device performing the perforation?

Schäfer: You are correct in assuming that the paper on which the hydrograph is recorded must have a perforation at its edge, as is also the case with paper tape. We do not have a device for automatically providing this perforation. It is also necessary to have the 4- or 5-week, and sometimes even 6-week, sheets joined together. For this reason we first transfer sheet records to tape, as explained already.

Herfst: For the processing of your charts you need a perfect curve without missing data and extremes, because the machine cannot give its own interpretation. The corrections have to be done manually. How many charts need such a correction before they can be processed?

Schäfer: Naturally, the machine cannot decide whether or not stage value has been recorded correctly. For this reason, the records must first be corrected by hand.

Ganahl: Can reversals be evaluated by your instrument? If so, in which order?

Schäfer: That is a question of programming. Our machine is capable of correctly evaluating reversal records, i.e., curves that start again at zero. Then, the first range (5 m at the 20 : 1 scale = 25 cm) is added. One could of course, also programme the machine so that the so-called reversing recorders could also be scanned properly. This, however, has not been done in the case of our machine.

Ganahl: You spoke of a stepwise evaluation of the gauged water level. Is this level immediately converted into discharge values and integrated, or is this done only after the daily mean of the water stage has been determined? If the latter were the case, the discharge value would include errors in the order of up to 50 per cent. This high percentage was discovered to appear in the evaluation of gauge readings taken at the downstream side of power stations.

Schäfer : Our machine does not convert values. As I have said already, it gives the daily means of the water stages, the monthly mean, and also the two extreme values. Further processing, such as the determination of discharge values, must then be effected with the aid of a variable-programme computer.

Behrens: (a) In comparison with the subsequent evaluation by computer, the scanning of a recorder strip requires a relatively long time. Do you believe that a development is technically feasible which would provide for the data being recorded at the gauge directly on data carriers that can be used in a computer?

(b) I am thinking of the possibility of equipping the conventional recording gauge with a sequential potentiometer and of having the data stored in a suitably connected, simple magazine-type magnetic-tape recorder. The records can be easily checked by printing out the stage hydrograph recorded on the magnetic tape, and comparing it with the directly plotted line.

(c) As a disadvantage of the direct measurement of tracers in the field, you mentioned the fact that the measuring instrument is bound to one measuring point. What we did for the control of intermixture was to direct the samples, extracted by pumping from five points of a cross-section, to a measuring instrument by means of an automatic arrangement. This more intensive utilization of the measuring instrument is, of course, possible only if the measuring points are located close to one another.

Schäfer : (a) I could imagine that the developement will gain in importance in the future. The punched tape has advantages and disadvantages. Its greatest disadvantage is probably the fact that it cannot be used at all existing gauging stations because these stations are often in a very poor condition. Once the curve has been recorded and plotted, it is comparatively easy to check visually and make corrections. This is not quite as simple with computer-type data carriers.

(b) This is a question of what one is willing to invest. If the graphic representation must be produced anyway, the other arrangement would be only an expedient providing for a perhaps more comfortable way of evaluation. But, as you have heard already, there exist nowadays semi-automatic and automatic devices which effect the conversion later on without first making this detour.

The intense evaluation of discharge measurements by the equations of the universal velocity distribution law

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Abstract. The most recent investigations of the author show that the assumptions made in obtaining the universal velocity distribution laws are valid also for natural channels. These findings are the basis for the evaluation of discharge measurements by means of the velocity distribution laws.

With the help of the "wall law", the turbulent shear velocities v_* and the effective roughness k_S may be computed. The value k_S represents the total hydraulic roughness produced by turbulent friction and further additional energy losses. The additional determination of the mean shear velocity $\overline{v}_*$ of the cross-section will provide the initial data for the computation of the mean velocities u and $\overline{u}$ of each vertical and the total cross-section. By means of the sectional area F and the velocity u, the total discharge Q of the cross-section is determined ($Q = F \cdot \overline{u}$).

Compared with the conventional method this procedure for evaluation discharge Q also permits the determination of the hydraulic parameters v_*, $\overline{v}_*$, k_S and $\overline{k}_S$.

EVALUATION INTENSIVE DES MESURES DE DEBITS A L'AIDE DES LOIS UNIVERSELLES DE DISTRIBUTION DES VITESSES

Résumé. Les résultats obtenus dans des recherches récentes de l'auteur ont révélé que les lois de répartition logarithmique des vitesses peuvent être étendues – sous certaines conditions – aux canaux naturels. Cette découverte fournit la base d'une nouvelle méthode d'évaluation du débit.

Les calculs commencent sur les verticales. A l'aide de la "loi de paroi" de Prandtl, il devient possible de déterminer les vitesses de cisaillement v_* et la rugosité hydraulique totale causée par le frottement en régime turbulent et par d'autres pertes de l'énergie. La détermination additionnelle de la vitesse de cisaillement $\overline{v}_*$ de la section mouillée fournit les données de base pour la détermination des vitesses moyennes u et $\overline{u}$ de chaque verticale et de la section mouillée.

A l'aide de la section F et de la vitesse $\overline{u}$, on détermine le débit total Q ($Q = F \cdot \overline{u}$).

Cette méthode d'évaluation du débit Q – comparée à la méthode conventionnelle – offre également l'avantage d'une détermination additionnelle des paramètres hydrauliques v_*, $\overline{v}_*$, k_S et $\overline{k}_S$.

EVALUACIÓN DE MEDIDAS DE CAUDAL POR LAS ECUACIONES DE LA LEY GENERAL DE DISTRIBUCIÓN DE VELOCIDADES

Resumen. Las investigaciones más recientes del autor revelaron que las ecuaciones de la ley de distribución de velocidades, haciendo determinadas hipótesis, son válidas también para los cauces naturales. Estos descubrimientos son la base de la evaluación de los caudales por medio de las leyes de distribución de velocidades.

El cálculo comienza en las verticales. Con ayuda de la ley de Pradtl se pueden calcular las velocidades de movimiento turbulento v_* y la rugosidad efectiva total k_S. La magnitud k_S representa la rugosidad hidráulica total producida por la fricción turbulenta y otras pérdidas de energía adicionales. La determinación adicional de la velocidad media $\overline{v}_*$ de la sección transversal proporcionará los datos iniciales para el cálculo de las velocidades medias $\overline{u}$ y $\overline{u}$ de cada vertical y de la sección transversal total. Por medio del área de la sección F y de la velocidad $\overline{u}$ se determina el caudal total Q de la sección transversal ($Q = F \cdot \overline{u}$).

Comparado con el método convencional este procedimiento para la evaluación del caudal Q permite además la determinación de los parámetros hidráulicos v_*, $\overline{v}_*$, k_S y $\overline{k}_S$.

ДЕТАЛЬНАЯ ОЦЕНКА ИЗМЕРЕНИЙ РАСХОДА С ПОМОЩЬЮ УНИВЕРСАЛЬНОГО ЗАКОНА РАСПРЕДЕЛЕНИЯ СКОРОСТИ

Резюме. Недавние исследования автора обнаружили, что уравнения универсального распределения скорости по вертикали, если сделать некоторые допущения, также применимы для естественных русел; эти выводы основаны на анализе измерений расхода посредством законов распределения скоростей.

Вычисления начинаются на вертикалях. С помощью «пристенного закона» подсчитываются скорости турбулентного сдвига v_* и эффективная шероховатость k_s. Величина k_s характеризует полную гидравлическую шероховатость, производимую турбулентным трением и другими дополнительными потерями энергии. Дополнительное определение средней скорости сдвига v_* поперечного сечения должно дать начальные данные для подсчета средных скоростей u и u на каждой вертикали и всего поперечного сечения. С помощью площади сечения F и скорости u полный расход Q поперечного сечения определяется, как обычно ($Q = F \cdot u$).

Сравнение этого метода расчета расхода Q со стандартными методами показывает, что он к тому же дает возможность определить гидравлические параметры v_*, v_*, k_s и k_s.

1. INTRODUCTION

The universal laws of logarithmic velocity distribution were developed theoretically by Prandtl (1925, 1932, 1933) and Kármán (1920, 1921) in the years 1926-30. Nikuradse (1926a, 1926b, 1930, 1933) verified these theories by means of his famous experiments for pipes in 1932/33. Later on Schlichting (1936), Keulegan (1938) and other investigators confirmed the logarithmic velocity distribution in artificial open channels.

Quite recent studies by the author show that the laws of logarithmic velocity distribution are valid also for rivers (Unbehauen, 1970). The above result is the basis for a new method for evaluating discharge from flow measurements which will be described here.

An appendix shows the evaluation of the discharge measurement for a gauging station.

2. THE UNIVERSAL LAWS OF LOGARITHMIC VELOCITY DISTRIBUTION

2.1. General assumptions

The universal laws of logarithmic velocity distribution are directly deduced from the "mixing length theory" by Prandtl (1925). Another approach is possible by means of the "similarity hypothesis" after Kármán (1920, 1921). Both derivations are described in detail by Schlichting (1964) and Unbehauen (1970).

Where natural flow conditions are dominant in rivers, the laws of velocity distribution for the turbulent region are of importance. Owing to this fact, the following considerations will be confined to completely rough channels.

2.2. The "wall law" of Prandtl

According to Figure 1 there is a relationship between the velocity u in the height y of the vertical Y and the turbulent shear-stress τ, which is expressed in the following equation:

$$u = 2.50 \sqrt{\frac{\tau}{\rho}} \cdot (\ln y + C). \tag{1}$$

This relationship represents the well known "wall law". The magnitude $\sqrt{\tau/\rho}$, having the dimension of a fictitious velocity, is generally termed "shear velocity". By substituting $v_* = \sqrt{\tau/\rho}$ in equation 1 and by using the decimal logarithm, equation 1 can be modified.

$$u = 5.75 \, . \, v_* \, (\log y + C'). \tag{2}$$

Equation 2 is most frequently used as the "wall law".

2.3. *The laws of logarithmic velocity distribution*

Considerations of the limiting values of the constants C and C' in equations 1 and 2 led Prandtl (1932) to the dimensionless equations of logarithmic velocity distribution:

$$\frac{u}{v_*} = 2.50 \, . \, \ln \frac{y}{k_s} + A \tag{3}$$

$$\frac{u}{v_*} = 5.75 \, . \, \log \frac{y}{k_s} + A'. \tag{4}$$

These relationships were verified experimentally by Nikuradse (1930a, 1933). The roughness height k_s characterizes the effective roughness, which exists in each vertical and is expressed as "equivalent sand roughness" after Schlichting (1936). The magnitude of the constant A' depends on the nature, height and spacing of the particles. Nikuradse found, for a uniform distribution of particles in a very close roughness spacing, that $A = 8.48$. Keulegan (1938) applied the equations 3 and 4 for open channels with non-circular cross-sections. He presupposes that supplementary losses can be neglected and that a uniform shear-stress distribution exists along the channel bottom. These assumptions make it possible to apply the friction velocity $v_{*w} = \sqrt{R \, . \, J \, . \, g}$ in which R is the hydraulic radius, J represents the slope of the energy gradient and g the acceleration of gravity in m/sec^2. By substituting v_{*w} in equation 4 Keulegan (1938) obtained:

$$\frac{u}{v_{*w}} = 5.75 \, . \log \frac{y}{k_s} + 8.50 \tag{5}$$

More recent investigations by Replogle and Chow (1966) and Unbehauen (1967) show that the assumptions of Keulegan are not generally valid.

In this paper the turbulent shear velocity $v_* = \sqrt{\tau/\rho}$ is used instead of the friction velocity $v_{*w} = \sqrt{h \, . \, J \, . \, g}$. This assumption is true for the region of fully developed turbulence. In this region the viscous forces are weak relative to the inertial forces.

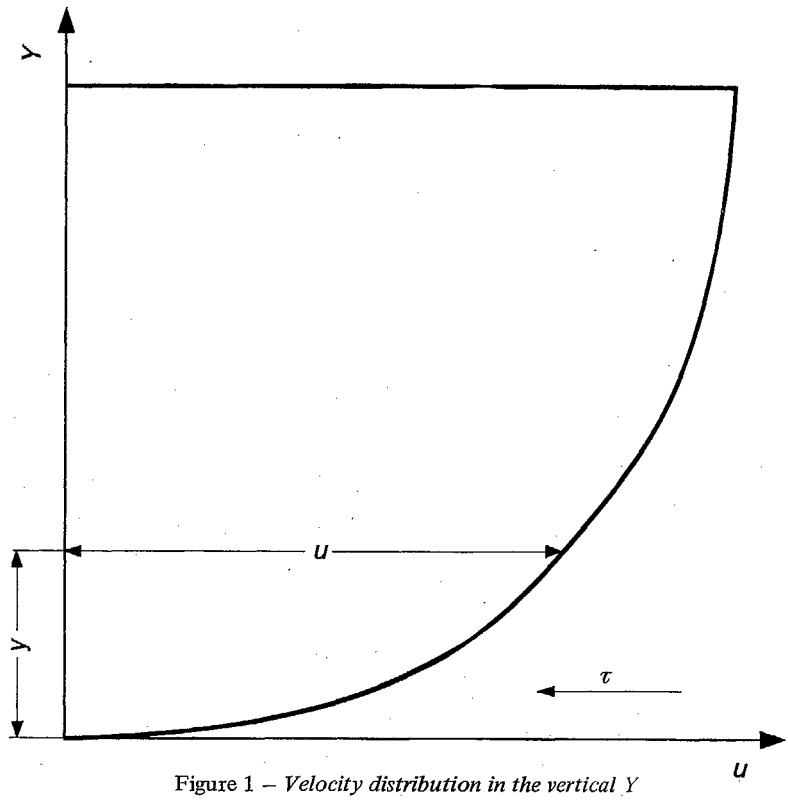


Figure 1 – *Velocity distribution in the vertical Y*

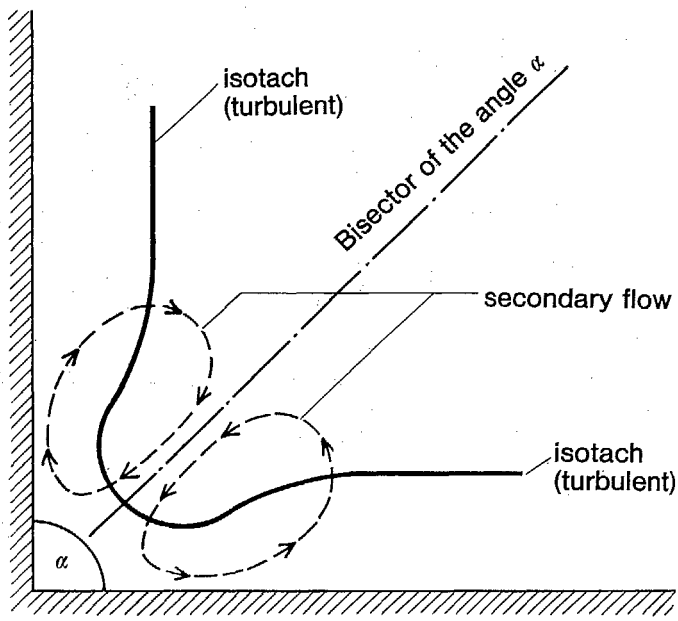


Figure 2 – *Isotach and secondary flow in a rectangular corner, after Gessner and Jones (1961)*

3. THE FLOW CHARACTERISTICS IN RIVERS

3.1. The additional head losses

In each channel there is a force resisting the flow of water. This force is caused by turbulent friction and additional head losses, originating from secondary currents, surface and local disturbances and from the shape of cross-sections.

3.2. Secondary flow

Secondary currents exist in each channel with noncircular cross-sections and in meandering rivers. The secondary flow is the movement of water particles in a cross-section normal to the longitudinal flow along the channel. This kind of current was discovered by Nikuradse (1930b) in smooth triangular, rectangular and narrowed circular pipes. Kirschmer (1949, 1952, 1952/53) found secondary currents in open cross-sections of rectangular and trapezoidal shapes.

For any internal corner of a cross-section (Fig. 2) there is a secondary flow into the corner along the bisector of the corner angle and away from the corner along the side walls. In general, secondary flow occurring whenever isotachs are curved is directed from the concave side toward the convex side of the isotachs (Fig. 3).

According to Kirschmer (1952/53) the intensity of secondary flow increases with the discrepancy of the individual cross-section from the shape of a circle. The formation of secondary currents is explained by several hypotheses described in detail by Unbehauen (1970).

Strong secondary currents dominate in meandering channels. After Raju (1932), it is assumed that in rectangular channels this kind of loss increases with the diminution of the factor r/b (r represents the radius of the bend and b the width of the channel). A strong influence on the amount of the head losses by secondary currents in meandering channels results from the ratio r/h. Optimal flow conditions exist when $r/h \approx 0.5 \div 0.75$. Decreasing ratios r/h result in greater losses because of secondary currents. These losses can approach the values of the frictional losses.

3.3. Surface losses

Surface losses caused by free surface turbulence exist near the water surface. The water level becomes very rough. The surface turbulence may be explained by the mixing length theory of Prandtl.

3.4. Local losses

Local losses are found near the boundaries where the roughness of the slope influences the flow. Natural slopes consisting of loose stones quite often show irregularities in the embankment lines. These slopes generally are covered by bushes and consequently have a more effective roughness than the bottom. Local losses may be caused also by local secondary flows or eddy currents near the bottom.

3.5. Shape losses

Figure 4 shows three equal cross-sectional areas with different depths h_1, h_2 and h_3. Provided that the "wall law" is valid in these cross-sections, different velocities occur. Thus there are three different discharges in the three cross-sections of equal areas. The half-circular cross-section carries away the optimal discharge. The more the

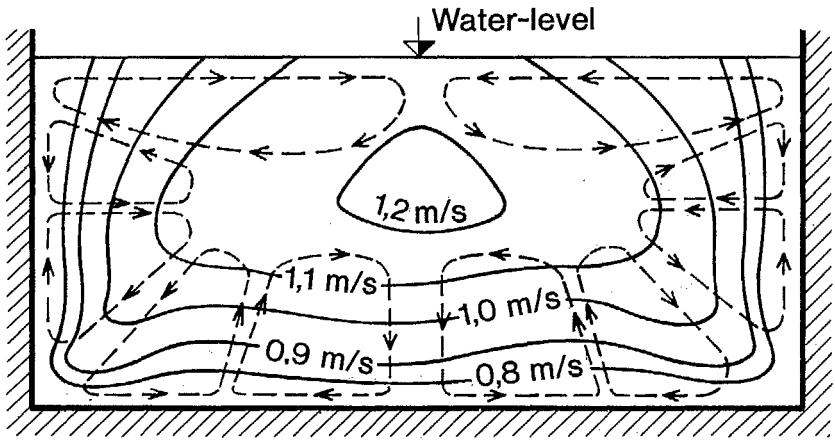


Figure 3 – *Velocity distribution and secondary currents in an open rectangular channel, after Kirschmer (1952)*

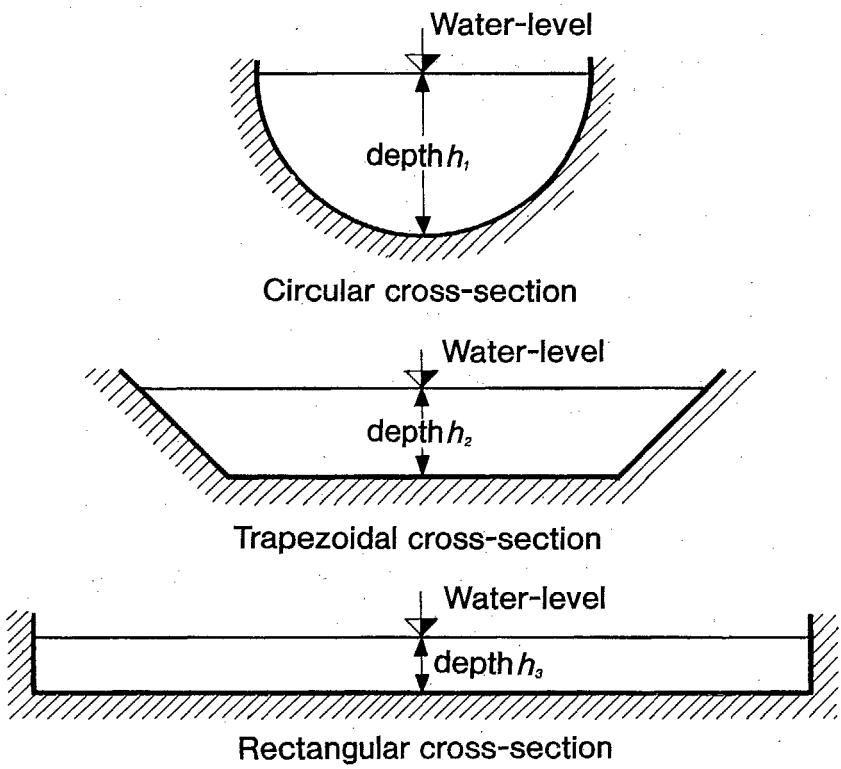


Figure 4 – *Different shapes of cross-sections with equal areas and different depths* $h_1 > h_2 > h_3$

shape of cross-section differs from the circular form, the smaller will be the discharge for equal cross-sectional areas. The additional losses developing in this way are called "shape losses".

4. THE APPLICATION OF THE LAWS OF LOGARITHMIC VELOCITY DISTRIBUTION

The practical studies of the author start with the equations 2 and 4.

4.1. The evaluation of the shear velocity v by means of the "wall law".

The basis of the evaluation is the "wall law" of Prandtl (1932, 1933), represented by equation 2:

$$u = 5.75\, v_* \,(\log y + C') \tag{2}$$

$$= C_2 \,.\, \log y + C_1 . \tag{6}$$

According to Figure 5, in the vertical Y the velocities $u_1, u_2, u_3 \ldots u_i, u_0$ were measured at heights $y_1, y_2, y_3, \ldots y_i, h$ above the bottom. The values obtained are plotted in a system of co-ordinates as shown in Figure 6. The X axis represents the velocities $u_1, u_2, u_3 \ldots u_i, u_0$; in the direction of the ordinate, the logarithms of the heights $y_1, y_2, y_3 \ldots y_i, h$ are plotted. The co-ordinates u_i, y_i are placed near the straight line having the equation

$$\begin{aligned} u &= C_1 + C_2 \,.\, \log y \\ &= C_1 + 5.75\, v_* \log y \end{aligned} \tag{7}$$

corresponding to the "wall law" in the form of equations 6 and 2. The magnitude of the shear velocity v_* can be deduced from the following equation:

$$v_* = \frac{C_2}{5.75} \,. \tag{8}$$

Numerous measurements at gauging stations with highly different runoff characteristics show all measured values are grouped along straight lines characterized by equation 7.

4.2. *The evaluation of the effective roughness k_s of each vertical*

After Nikuradse (1933) for the "natural sand roughness k_s" we have:

$$\frac{u}{v_*} = 5.75 \,\log \frac{y}{k'_s} + 8.50. \tag{4}$$

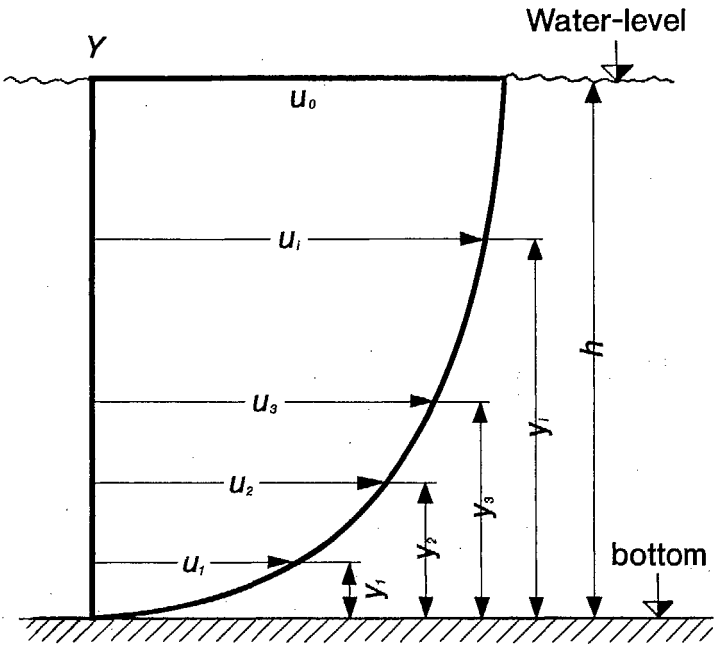


Figure 5 – *The measured velocities in the vertical Y (scheme)*

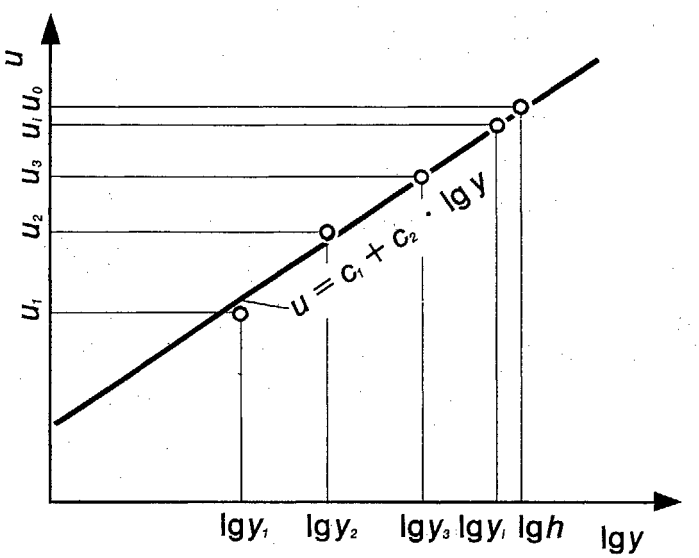


Figure 6 – *The "wall law" for the vertical Y of Figure 5 (scheme)*

In a later equation k_s' expresses the effective roughness of a vertical evaluated from one measurement. The following transformations of equation 2 result in:

$$\frac{u}{v_*} = 5.75 \log y + C'$$
$$C' = \frac{C_1}{v_*}. \tag{9}$$

A comparison of equations 4 and 9 leads to the following expression:

$$\log k_s' = \frac{8.50 - C'}{5.75}. \tag{10}$$

By means of relationship 10, it is possible to determine the effective roughness k_s' of each measurement, which may be averaged to give the effective roughness k_s of n measurements: $k_s = \frac{\Sigma k_s'}{n}$.

4.3. *The determination of the mean velocities u in the verticals and the mean velocities u of the cross-section*

The calculation begins with equation 3:

$$u = v_* \left(2.50 \,.\, \ln \frac{y}{k_s} + A\right) \tag{3}$$

by integrating and averaging, equation 3 can be transformed to:

$$\bar{u}' = \frac{1}{h} \int_{y=0}^{y=h} v_* \left(2.50 \,.\, \ln \frac{y}{k_s} + \mathrm{A}\right) \mathrm{d}y$$

and

$$\bar{u}' = v_* \left(2.50 \,.\, \ln \frac{h}{k_s} + \mathrm{A} - 2.50\right) \tag{11}$$

$$\bar{u}' = v_* \left(5.75 \log \frac{h}{k_s} + \mathrm{A} - 2.50\right) \tag{12}$$

$$\bar{u}' = v_* \left(5.75 \log \frac{h}{k_s} + \bar{A}'\right). \tag{13}$$

Equation 13 will be used to compute the mean velocity $\bar{u}'$ of a vertical. By introducing the mean magnitudes of the cross-section

$$u\,,\ \bar{v}_* = \frac{\Sigma (h \,.\, v_*)_i}{\Sigma h_i}\ ;\ \left(\frac{h}{k_s}\right) = \frac{1}{b} \cdot \sum_0^b \left(b \,.\, \frac{h}{k_s}\right),$$

and $\bar{A} = \bar{A}' = 6.00$ in equation 13 we find:

$$\bar{u} = \bar{v}_* \cdot \left[5.75 \cdot \log \left(\frac{h}{k_s} \right) + \bar{A} \right]. \tag{14}$$

Equation 14 represents the basic relationship for determining the mean velocity $\bar{u}$ of the cross-section.

4.4. *The computation of the discharge Q in the cross-section F*

The discharge Q of the cross-section F will be easily obtained from equation 15:

$$Q = F \cdot \bar{u}. \tag{15}$$

BIBLIOGRAPHY / BIBLIOGRAPHIE

Chow, V. T. *Open-Channel-Hydraulics.* New York, McGraw-Hill Book Comp. Inc.

Gessner, F. B.; Jones J. B. 1961. A preliminary study of turbulence characteristics of flow along a corner. *Journal of Basic Engineering-Transactions of the ASME.*

Kármán, T. V. 1921. Über laminare und turbulente Reibung. *Zeitschr. f. angew. Math. u. Mech.*, vol. 1.

—. 1930. Mechanische Ähnlichkeit und Turbulenz. Nachr. d. Gesellsch. d. Wissensch. zu Göttingen. Göttingen 1920. Verhandl. d. 3. Internat. Kongresses f. techn. Mechanik in Stockholm.

Keulegan, G. H. 1938, Laws of turbulent flow in open channels. *Journal of Research of the National Bureau of Standards,* vol. 21, paper no. 1151.

Kirschmer, O. 1949. Reibungsverluste in Rohren und Kanälen. *Deutsche Wasserwirtschaft Jahrg.*, no. 7/8.

—. 1952. Kritische Betrachtungen zur Frage der Rohrreibung. *VDI-Zeitschr.*, vol. 94, no. 24.

—. 1952/53. Zur Frage einheitlicher FlieB formeln für offene Gerinne. Die Wasserwirtschaft, 146 p.

Nikuradse, J. 1926a. Untersuchung über die Geschwindigkeitsverteilung in turbulenten Strömungen. *VDI-Forschungsheft,* 281.

—. 1926b. Geschwindigkeitsverteilung in turbulenten Strömungen. *VDI-Zeitschr.*, vol. 70.

—. 1930a. Widerstandsgesetz und Geschwindigkeitsverteilung von turbulenten Wasserströmungen in glatten und rauhen Rohren. Verhandl. d. 3. intern. Kongresses für techn. Mechn. in Stockholm.

—. 1930b. Untersuchungen über turbulente Strömungen in nichtkreisförmigen Rohren. *Ing.-Arch.*

—. 1933. Strömungsgesetze in rauhen Rohren. *VDI-Forschungsheft,* 361, July/Aug.

Prandtl, L. 1925. Über die vollausgebildete Turbulenz. *Zeitschr. f. ang. Math. u. Mech.*, vol. 5.

—. 1932. Zur turbulenten Strömung in Rohren und längs Platten. *Ergebnisse d. Aerodyn. Versuchsanstalt,* Göttingen.

—. 1933. Neuere Ergebnisse der Turbulenzforschung. *VDI-Zeitschr.*, vol. 77, no. 5, p. 106.

Raju, S. B. 1932. Versuche über den Strömungswiderstand gekrümmter offener Kanäle. *Mitt. d. Hydraul. Inst. der T. H. München,* no. 6.

Replogle, J. A.; Chow, V. T. 1966. Tractive force distribution in open channels. *Proceed. A.S.C.E. Journal of Hydro. Div.*, no. HY2.

Schlichting, H. 1936. Experimentelle Untersuchungen zum Rauhigkeitsproblem. Sonderdruck Ing. – Arch., vol. VII, no. 1, Febr.

—. 1964. Grenzschichttheorie. Verlag C. Braun Karlsruhe.

Unbehauen, W. 1967. Gedanken zur Verwendung der Darcy'schen Strömungsgleichung in nichtkreisförmigen geschlossenen und offenen Querschnitten. Diss. T. U. Hannover.

—. 1970. Die universelle logarithmische Geschwindigkeitsverteilung in natürlichen Gerinnen. Schriftenreibe d. Bayer. Landesstelle für Gewässerkunde, no. 2.

APPENDIX. *Evaluation of the discharge measurement of the gauging station Wasserburg/Inn (F_n = 11,983 km²) on 4 July 1969; water-level w = 261 cm*

7.1. Table A. *Computation of v_* and k'_s (For notes (1), (2), (3) please see page).*

Vertical $x = \ldots m$(1) (m)	measured y (cm)	log y	measured u (cm/sec)	C_2 (2) (cm/sec)	$v_* = \frac{C_2}{5.75}$ (cm/sec)	C_1 (2) (cm/sec)	$C' = \frac{C_1}{v_*}$ (2)	$\log k_s = \frac{8.50 - C'}{5.75}$	k'_s (cm)
5	21	1.322	17	71.0	12.34	−71.5	−5.75	2.478	300(3)
	43	1.633	40						
	70	1.845	54						
	96	1.982	72						
	122	2.086	82						
	149	2.173	89						
	175	2.243	103						
10	20	1.301	33	71.0	12.33	−57.5	−4.66	2.290	195(3)
	85	1.929	75						
	153	2.185	93						
	221	2.344	109						
	288	2.459	119						
	356	2.551	136						
	424	2.627	136						
	492	2.692	136						
15	21	1.322	54	58.0	10.08	−22.0	−2.18	1.855	71.6
	95	1.978	86						
	173	2.238	112						
	251	2.400	122						
	328	2.516	130						
	406	2.609	133						
	484	2.685	128						
	562	2.750	133						
20	20	1.301	52	58.0	10.08	−22.0	−2.18	1.855	71.6(3)
	94	1.973	94						
	170	2.230	94						
	247	2.393	119						
	323	2.509	124						
	400	2.602	138						
	476	2.678	145						
	553	2.743	145						
25	21	1.322	74	54.0	9.38	−1.5	−0.16	1.505	32.0
	90	1.954	94						
	164	2.215	105						
	237	2.375	133						

Vertical $x = \ldots m$(1) (m)	measured y (cm)	log y	measured u (cm/sec)	C_2(2) (cm/sec)	$v_* = \frac{C_2}{5.75}$ (cm/sec)	C_1(2) (cm/sec)	$C' = \frac{C_1(2)}{v_*}$	log $k_S' = \frac{8.50 - C'}{5.75}$	k_S' (cm)
25	311	2.493	131	54.0	9.38	− 1.5	− 0.16	1.505	32.0
	384	2.584	142						
	458	2.661	140						
	531	2.725	140						
30	26	1.415	49	70.0	12.17	− 40.0	− 3.28	2.045	111
	88	1.944	93						
	158	2.199	93						
	229	2.360	135						
	299	2.476	147						
	370	2.568	151						
	440	2.643	152						
	511	2.708	147						
35	21	1.322	93	62.0	10.78	6.0	0.56	1.382	24.1
	78	1.892	122						
	138	2.140	138						
	199	2.299	145						
	260	2.415	158						
	321	2.507	159						
	381	2.581	163						
	442	2.645	156						
40	21	1.322	94	63.5	11.04	6.2	0.56	1.382	24.1
	124	2.093	138						
	184	2.265	151						
	245	2.389	149						
	306	2.486	158						
	367	2.565	151						
	427	2.630	149						
	488	2.688	152						
45	21	1.322	88	63.5	11.04	4.7	0.43	1.400	25.1
	88	1.944	130						
	160	2.204	147						
	231	2.364	149						
	302	2.480	168						
	373	2.572	170						
	445	2.648	172						
	516	2.713	177						
50	21	1.322	114	64.0	11.13	14.0	1.25	1.260	18.2
	85	1.929	143						

Vertical $x = \ldots m$[1] (m)	measured y (cm)	$\log y$	measured u (cm/sec)	C_2[2] (cm/sec)	$v_* = \frac{C_2}{5.75}$ (cm/sec)	C_1[2] (cm/sec)	$C' = \frac{C_1(2)}{v_*}$	$\log k_s' = \frac{8.50 - C'}{5.75}$	k_s' (cm)
50	153	2.185	143	64.0	11.13	14.0	1.25	1.260	18.2
	220	2.342	158						
	288	2.459	163						
	356	2.551	156						
	423	2.626	170						
	491	2.691	164						
55	24	1.380	114	63.0	10.95	4.5	0.41	1.405	25.4
	77	1.886	112						
	136	2.134	142						
	196	2.292	145						
	255	2.407	158						
	315	2.498	158						
	374	2.573	159						
	434	2.637	173						
60	21	1.322	101	73.0	12.67	8.5	0.67	1.363	23.1
	67	1.826	128						
	117	2.068	166						
	167	2.223	182						
	218	2.338	185						
	268	2.428	182						
	318	2.502	185						
	368	2.566	187						
65	22	1.342	119	71.0	12.37	7.5	0.61	1.372	23.6
	59	1.771	130						
	102	2.009	152						
	144	2.158	161						
	187	2.272	164						
	229	2.360	177						
	272	2.435	170						
	314	2.497	170						
70	20	1.301	94	69.0	12.00	7.0	0.58	1.375	23.7
	61	1.785	136						
	106	2.025	147						
	150	2.176	163						
	194	2.288	168						
	239	2.378	175						
	283	2.452	173						

Vertical $x = \ldots m$[1] (m)	measured y (cm)	$\log y$	measured u (cm/sec)	C_2[2] (cm/sec)	$v_* = \frac{C_2}{5.75}$ (cm/sec)	C_1[2] (cm/sec)	$C' = \frac{C_1(2)}{v_*}$	$\log k_s' = \frac{8.50 - C'}{5.75}$	k_s' (cm)
75	23	1.362	105	64.5	11.22	5.7	0.51	1.390	24.6
	54	1.732	121						
	91	1.959	131						
	129	2.111	152						
	166	2.220	149						
	203	2.308	152						
	240	2.380	157						
80	21	1.322	107	73.0	12.69	6.5	0.51	1.390	24.6
	48	1.681	128						
	78	1.892	145						
	109	2.037	163						
	140	2.146	151						
	170	2.230	166						
	201	2.303	161						
85	21	1.322	75	70.0	12.16	−15.0	−1.23	1.692	49.2
	43	1.633	98						
	70	1.845	124						
	96	1.982	119						
	122	2.086	136						
	149	2.173	147						
	175	2.243	147						
90	23	1.362	72	62.0	10.79	−13.0	−1.21	1.686	48.5
	38	1.580	79						
	59	1.771	105						
	80	1.903	107						
	100	2.000	109						
	121	2.083	116						
	142	2.152	112						
95	22	1.342	26	42.0	7.30	−26.0	−3.56	2.100	126[3]
	31	1.491	38						
	45	1.653	49						
	60	1.778	47						
	74	1.869	49						
	88	1.944	47						
	102	2.009	49						
	116	2.064	38						

Vertical $x = \ldots m$[1] (m)	measured y (cm)	$\log y$	measured u (cm/sec)	C_2[2] (cm/sec)	$v_* = \frac{C_2}{5.75}$ (cm/sec)	C_1[2] (cm/sec)	$C' = \frac{C_1(2)}{v_*}$	$\log k_s' = \frac{8.50 - C'}{5.75}$	k_s' (cm)
100	21	1.322	44	70.8	12.31	−54.6	−4.46	2.250	178[3]
	30	1.477	47						
	37	1.568	49						
	44	1.643	61						
	51	1.708	65						
	58	1.763	70						
	65	1.813	66						

Notes

(1) Vertical in the distance x from the fixed point at the left bank.

(2) C_1 and C_2 by equation 3 : $u = 5.75\ v_* (\log y + C')$

$$= C_2 \log y + C_1 \text{ ; whereby } C' = \frac{C_1}{v_*} \text{ and } v_* = \frac{C_2}{5.75}.$$

Relation 3 represents the equation of the arranged straight-line in the co-ordinates (u , $\log y$) (Fig. 6). The arranged straight-line is obtained by computation or graphical solution.

(3) k_s' influenced by additional local losses in the vicinity of the natural slopes.

7.2. Table B. *Computation of* $\left(\frac{\bar{h}}{k_s'}\right)$, $\bar{k}_s$, $\bar{u}_{ger}$ *and* $\bar{v}_*$

Vertical $x = \ldots m$(1) (m)	measured h (cm)	k_s' (cm)	$\frac{h}{k_s'}$	$\left(\frac{\bar{h}}{k_s'}\right)$	$\bar{A}' = A - 2.50$	$\bar{v}_*$ (cm/sec)	computed $\bar{u}'$(2) (cm/sec)	measured $\bar{u}$ (cm/sec)	$h \cdot v_*$ (cm/sec)	$\bar{v}_*$ (cm/sec)
1	0	0	0			0	0	0	0	0
5	182	300	0.61	10.16	6.00	12.34	59	62	2245	11.10
10	499	195	2.56		6.00	12.33	103	106	6140	
15	569	71.6	7.95		6.00	10.08	113	113	5740	
20	560	71.6	7.83		6.00	10.08	113	115	5640	
25	538	32.0	16.85		6.00	9.38	123	120	5060	
30	518	111	4.66		6.00	12.17	120	121	6300	
35	449	24.1	18.65		6.00	10.78	143	142	4810	
40	495	24.1	20.50		6.00	11.04	149	141	5450	
45	523	25.1	20.80		6.00	11.04	150	150	5780	
50	498	18.2	27.30		6.00	11.13	158	152	5530	
55	441	25.4	17.30		6.00	10.95	144	144	4800	
60	375	23.1	16.22		6.00	12.67	164	164	4750	
65	321	23.6	13.60		6.00	12.37	154	154	3960	
70	290	23,7	12.23		6.00	12.00	147	150	3480	
75	247	24.6	10.04		6.00	11.22	133	135	2770	
80	208	24.6	8.45		6.00	12.69	144	142	2640	
85	182	49.2	3.70		6.00	12.16	113	117	2210	
90	149	48.5	3.07		6.00	10.79	95	95	1610	
95	123	126	0.98		6.00	7.30	43	39	897	
100	72	178	0.40		6.00	12.31	46	52	885	
104.8	0		0			0	0	0	0	

Results :

$\bar{k}_s' = 41$ cm;

$\bar{v}_* = 11.10$ cm/sec;

Computed $\bar{u} = \bar{v}_*\left(6.00 + 5.75 \log\left(\frac{\bar{h}}{k_s'}\right)\right)$

$= 11.10(6.00 + 5.75 \log 10.16)$

$= 131$ cm/sec;

Measured $\bar{u} = 129$ cm/sec;

Computed Q = comp. $\bar{u} \cdot F = 1.31 \cdot 360.7$

$= 471$ m^3/sec;

Measured $Q = 466$ m^3/sec;

$\frac{\bar{h}}{k_s'} = \frac{398}{41} = 9.70$

$\left(\frac{\bar{h}}{k_s'}\right) = \frac{\Sigma\left(\frac{h}{k_s'}\right)_i \cdot b_i}{b} = 10.16$

$\beta = \left(\frac{\bar{h}}{k_s'}\right) : \frac{\bar{h}}{k_s'} = 1.05$.

Notes

(1) Vertical in the distance $x(m)$ from the fixed point at the left bank

(2) Computed $\bar{u}' = v_*\left(A + 5.75 \log \frac{\bar{h}}{k_s'}\right)$ after equation 13.

DISCUSSION

De Vries: The determination of the roughness from velocity profiles will fail if ripples and dunes are present, simply due to the fact that the roughness is not uniformly distributed.

Unbehauen: I should accept this argument if I had based the method on the roughness of the whole cross-section. However, the method is based on the determination of the roughness point by point, leading finally to the total roughness of the cross-section. Any peculiarities in the shape or roughness at a given point in the cross-section will, therefore, influence the result. We have evaluated about fifty measurements in completely different rivers, mountainous rivers with a slope of 5 per cent and rivers in the plain with a slope of only 0.1 per thousand. The results were good. In comparison with the conventional methods, we had differences of ± 2 per cent and I should like to leave it open regarding making a decision as to which method is the more appropriate one.

De Vries: Did you have ripples and dunes?

Unbehauen: The method is based not only on the wall friction but also on friction by turbulence. All losses are contained in it. The influences of the bottom, however, are not as strong as those from the turbulence.

Oebius: The logarithmic law of distribution of velocity has been established for a clearly defined flow and therefore, cannot be used in nature without reservations. The influence of various coefficients of roughness is not contained in this law, in spite of their possible importance. The straight line in the logarithmic paper is nice, but must not necessarily correspond to reality. The results will be doubtful if the values C_1 and C_2 from the graph are used for the determination of k_s .

It is certainly possible to calculate the discharge with the help of this new method, if the circumstances are favourable and if the flow, cross-section and roughness are similar to the conditions used for the establishment of the law. I am afraid, however, that in nature changing roughness, changing cross-sections and changing turbulences will lead to wrong values.

Unbehauen: No, this value is actually not contained in the formula. The formula can be applied only in cases of full turbulence when the viscosity is much smaller in comparison with the inertia and can thus be disregarded. This is the basis of the calculation. The values near the bottom can, therefore, be neglected.

Oebius: No, you have to take it into account; it is extremely important. The distribution of velocity in the bordering layer differs completely from the cases described by Nikuradse or Prandtl for pipes. The velocity distribution assumed by Nikuradse and Prandtl for a rough surface refers to the logarithmic distribution, but you will admit that the difficulty is to find a constant value for the A in the law for logarithmic velocity distribution, and will agree that the influence of the bordering layer must be considerable.

Unbehauen: No, I do not quite agree. Factor A can be regarded as a constant value when k_s is introduced as the roughness coefficient according to Nikuradse. The value

of 8.5 mentioned in the paper depends on the distribution of roughness. It is, however, possible to transfer it into a sand roughness.

Gwinn: Did you use this method on a grass-lined channel?

Unbehauen: No.

Struijk: The logarithmic velocity law cannot be applied to all rivers. We have tried this same method in the International Standardization Organization measurements. But we only can say that sometimes it will hold and sometimes not. It depends entirely on the local circumstances of the measuring site.

On the discussion of Mr. Carter's article in the journal of the Hydraulics Division (A.S.C.E.), January 1964, you will see the same, especially the relative velocity is 0.4 of the relative depth.

Unbehauen: The logarithmic velocity distribution cannot be applied when the forces of viscosity must be taken into account vis-à-vis the force of the turbulence.

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