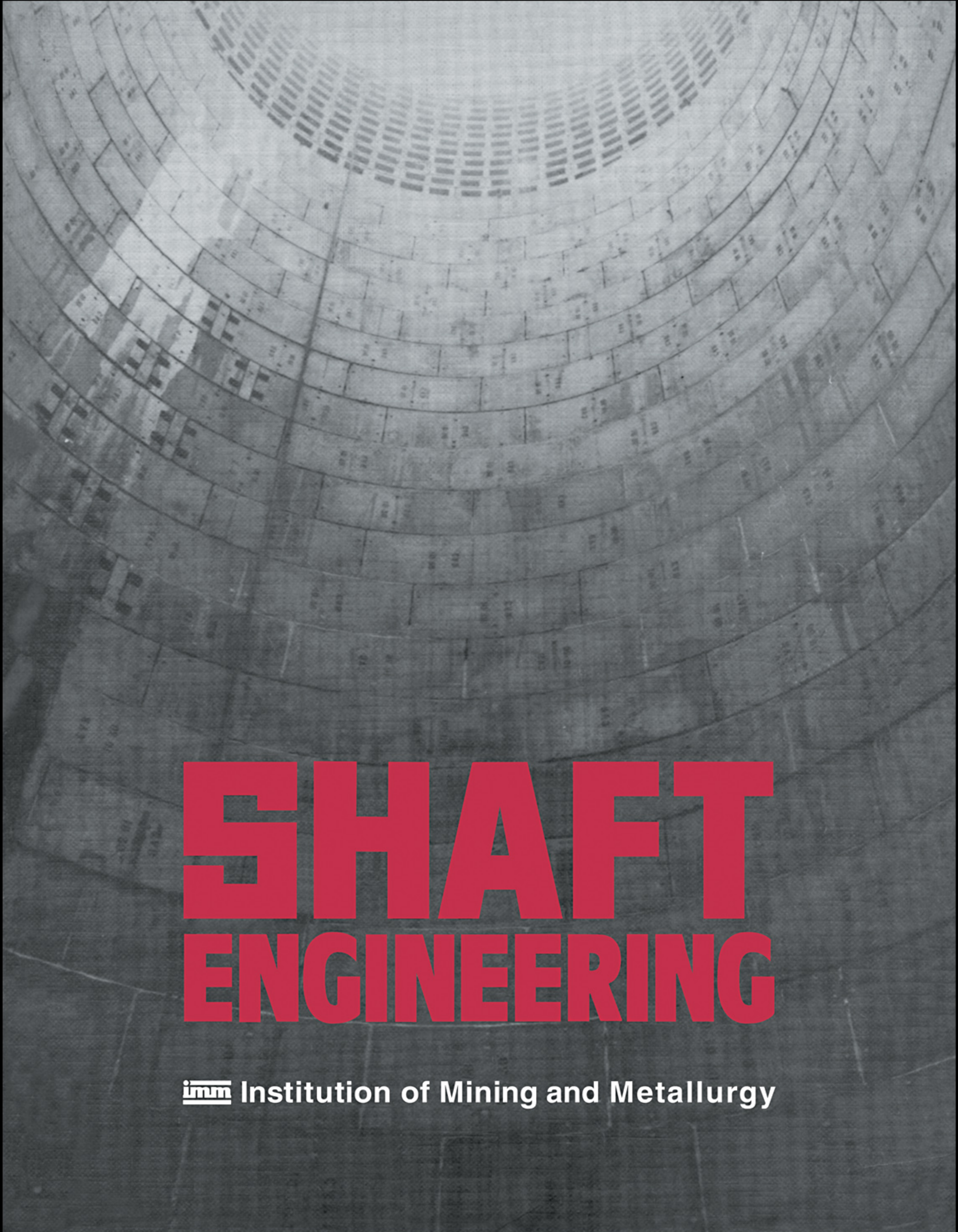

SHAFT ENGINEERING

Institution of Mining and Metallurgy

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SHAFT ENGINEERING

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Papers presented at the *Shaft Engineering* conference, organized by the Institution of Mining and Metallurgy in association with the Institution of Civil Engineers and the Institution of Mining Engineers, and held in Harrogate, England, from 5 to 7 June, 1989

Institution of Mining and Metallurgy

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Foreword

Although papers on shaft engineering have appeared from time to time in the *Transactions* of the Institution of Mining and Metallurgy and in its mining-related conference publications, the holding of a meeting that is devoted to the subject is overdue. There has been an excellent and truly international response to the invitation to submit papers for presentation and discussion at the June, 1989, 'Shaft engineering' conference—as is apparent in this volume, which contains 36 papers by authors from 13 countries.

The papers cover investigation, design and hydrogeology, shaft lining and furnishing, shafts for radioactive waste repositories, high-pressure shafts for hydroelectric schemes, shaft excavation, ground treatment and water control, shafts for civil engineering purposes and case-history descriptions. This range of topics bodes well for those who will be participating in the conference, offering a wealth of expertise and opportunity for informal discussions outside the technical sessions. But those who have to be content with reading this volume of papers will not, I trust, be disappointed with what amounts to a description of current activity and development in shaft engineering in the last years of the twentieth century.

To the authors I should express particular thanks—without their efforts there would be neither conference nor volume. Thanks are also due to my colleagues on the Organizing Committee, the referees of authors' manuscripts and Penny Gill and her staff at the Institution of Mining and Metallurgy—their endeavours are greatly appreciated.

L.R.Richards
Chairman
March 1989

Organizing Committee

Contents

Shaft drilling in the U.S.S.R.: history and recent experiences

A.Agarkow

New Drilling Technologies, Spezschachtoburenije SSB Donesk, U.S.S.R.

E.Leonenko

New Drilling Technologies, Spezschachtoburenije SSB Donesk, U.S.S.R.

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ABSTRACT

The paper covers the development of the soviet shaft drilling industry by recounting the use of rigs between 1936 and the present date. Special consideration is given to the rigs WIRTH type L 35 type rigs and major points from recent experience made during the last decade of drilling more than 25 shafts. The paper includes a description of the standard drilling and lining operations in use today and the performance achieved, including an account of a major extensive fishing operation.The aims of the Soviet Special Shaft Drilling Company together with WIRTH in developing a Steerable Drilling Device upon which the drilling industry could rely are explained. The first results from the industrial tests and applications of such a device are summarized.

1. INTRODUCTION

The Company Spezschachtoburenije (SSB), located in Donezk, USSR, was established in 1966 to meet the huge demand for shafts in the coal industry of the Soviet-Union and especially within the Ukraine for the coal mines of the Donezk area. From 1978, the year of the first direct contacts between SSB and WIRTH, a close cooperation and intimate relationship between both companies started to develop, fostered by the desire of both sides to overcome the obstacles arising from the transition from “big hole drilling” to “shaft drilling” and founded upon WIRTH’s deep involvement in developing machines for shaft drilling.

WIRTH started its manufacturing programme of shaft boring machines and shaft drilling rigs in 1964 by building its first Raise Boring Machine (fig. 1). Today, the Raise Boring programme of WIRTH covers all diameters up to 6 m and up to 1000 m depth. This programme was widened by developing the first Down Reaming Machine (fig. 2) built currently for diameters between 4.5 m and 8.20 m. These machines have been used on more than 40 shaft boring projects worldwide; totalling more than 12 km.

The maximum shaft dimensions achieved so far, are 8.20 m in diameter and 705 m in depth, both in boring the “Lummerschied” shaft in Gottelborn at Saarbergwerke/Germany. The development of a Fullface Shaft Boring Machine, with the cuttings being transported hydraulically to an upper level or to surface, was a logical step in a further extension of WIRTH’s shaft boring machine programme, and two practical tests in coal mines in Germany showed the feasibility of this concept (fig. 3); the last test at Heinrich Robert in 1983 was commented upon as follows:

“The Heinrich Robert Shaft has now been succesfully completed to a depth of 165 m (540 ft) in a drilling time of 64 days. Maximum advance was 7 m/d (23 ft per day) with an average of 2.5 m/d (8 ft per day) and 18 m/wk (59 ft per week).

While these average rates of advance are only two-thirds of those achieved in sinking the new Silver Shaft at Hecla’s Lucky Friday Mine, keep in mind that we are comparing the first use of a prototype machine with a record breaking performance by conventional methods. The job must be considered an outstanding success and foretells, in my opinion, the beginning of the mechanical shaft sinking area.” R.J.ROBBINS, Feb. 26–March 1, 1984

The manufacturing programme of WIRTH is completed by the shaft drilling rigs, type L, of which 14 rigs have been built during the last 10 years (fig. 4); 10 of them being more or less continuously in operation in the USSR.

The cooperation between Spezschachtoburenije, as the biggest shaft drilling contractor in the world, on one side, and WIRTH, the sole manufacturer currently producing a complete range of equipment for all shaft drilling and boring techniques, assures further developments helpful for the whole shaft drilling industry.

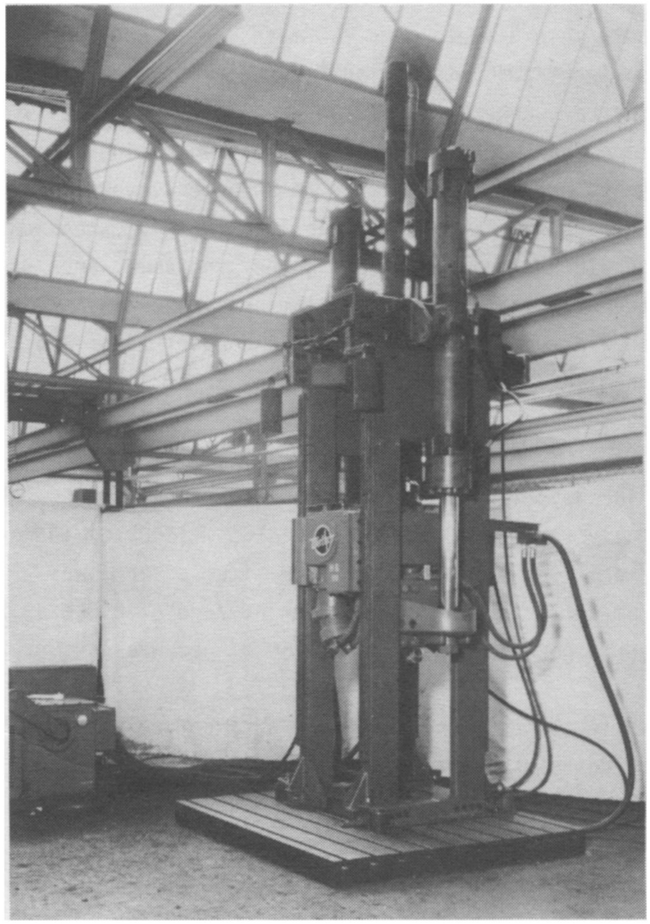


fig. 1: WIRTH raise boring machine type HG 170

2.
HISTORY OF SHAFT DRILLING IN THE USSR

Shaft drilling was used for the first time in the USSR, in 1938 by the utilisation of Rotary Drilling Rigs using reaming procedures. The max. dia meter drilled was 2.4 m; a pilot hole of 600 mm diameter being reamed in 6 steps of 300 mm each. The depth was between 30 and 100 m and 7 holes were drilled in total. After an interruption caused by the war, drilling with rotary rigs was started again, in 1947. In 1956, the first purpose-built shaft drilling rig, type UKB-3.6 was built and put into operation, as the modified oil field rigs previously used no longer met the requirements. This machine utilised coring techniques and recovered cores of 3 m diameter and 5 m in length weighing 100 tons, from a drilling diameter of 3.6 m. With this machine it was also possible for the first time to drill such a large hole in unstable rock by the ‘full-face’ drilling method. The UKB-3.6 drilled 3 shafts with a total depth of 1300 m; the max. individual depth being m, and the max. drilling performance was 153 m/month. At the 1958 World Exhibition in Brussels, the gold medal for construction was awarded to this shaft drilling rig. In the late fifties, development of the shaft drilling rigs USTM-6.2 and USTM-7.5/8.75 (fig. 5) was started, and they were put into operation in 1962, for the first time. This machine type drilled 4 shafts with diameters of 6.2 m and 7.5 m which are still used today as main shafts. The total depth of all 4 shafts was 1600 m.

At the same time, the so-called RTB-units (Re-action-Turbo Drills) equipped with components proved in the oil industry were put into operation. On the Reaction Turbo Drilling Rigs (fig. 6), up to 5 turbo-drills are mounted on a common support which is rotated by the reaction to their moments so that each turbo-drill has its own cutting track. This procedure is still widely employed in the USSR in spite of its limited technical possibilities. At the optimal drilling diameter of 2.4 m, a depth of 940 m has been achieved, the max. depth of 1220 m was drilled at 1.2 m diameter. The max. drilling diameter at the collar shaft is 5.5 m. At 3.2 m dia. 640 m depth was reached and at 4.0 m dia. 450 m depth. Between 1970 and 1980, 50 km of shaft were drilled in total by RTB rigs.

Since 1981, 10 air-lift drilling rigs L 35, L 35 K and L 35 M from WIRTH have been employed in the USSR with drilling diameters up to 4.7 m and a max. depth of 700 m with an inner shaft diameter of 3.5 m (fig. 7). Up to now, 25 shafts have been or are being drilled totalling to 9636 m, the deepest shaft achieved a final depth of 748 m.

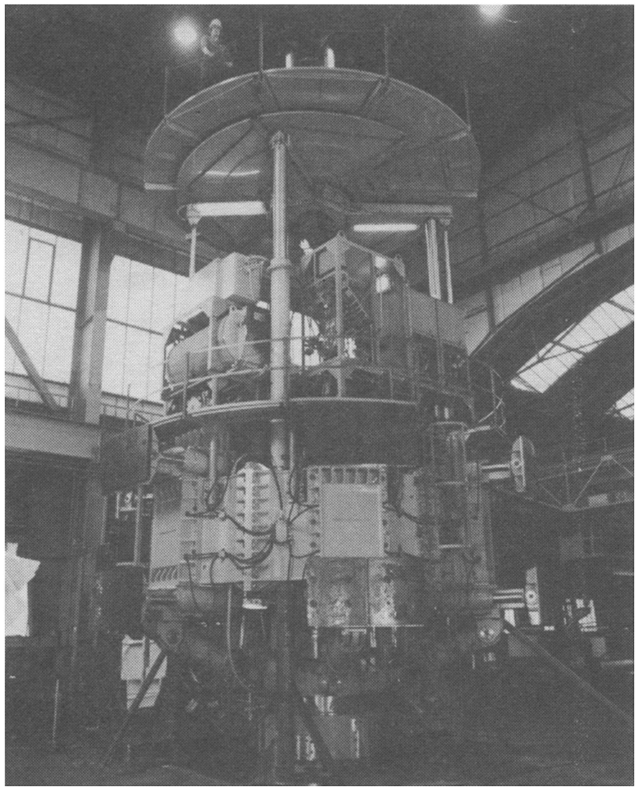


fig. 2: WIRTH down reaming machine type SB VII–650/850

Nowadays, these rigs are being utilised in the Ukraine, in the Rostow area, in Karaganda and in the area of Novokuznezk. The L 35 rigs were used in 1981 and 1982 for the first time, the L 35 K rigs in 1984 and 1985, whereas the three L 35 M rigs started drilling operations only in 1986 and 1987.

3. TECHNOLOGY OF SHAFT DRILLING

3.1 DRILLING EQUIPMENT

30 Reaction Turbo Drill Rigs (RTB) and 10 rotary Shaft Drilling Rigs of the WIRTH L 35, L 35 K and L 35 M type are used for drilling approx. 15 big holes or shafts a year, with an average depth of 500 m at diameters mostly between 3.6 to 4 m, corresponding to approx. 50000 to 55000 m^3 of excavation.

The WIRTH L 35 M type is the most modern shaft drilling rig built by WIRTH, in which all the experience gained from the use of 11 rigs in total in the PR of China and in the USSR (four L 40, three L 35 and four L 35 K) was been taken into consideration in modifications to the previous designs including most importantly,

- An extension of the derrick,
- The use of longer drill pipes (9 m and 12 m, instead of 4.5 m and 6 m respectively), reducing the drill string weight on one side and the time required for round trips on the other,
- The development of a drill pipe elevator for flanged pipe permanently installed between travelling block and swivel and which does not have to be specially mounted for round trips. Due to utilisation of the elevator, bolting and unbolting of the flange connection to the upper end is not necessary when adding or removing drill pipe. Operation of the elevator, such as opening, tilting and locking is effected, hydraulically. A pipe handler leads the drill pipe in a nearly horizontal attitude to the elevator which takes it over from the handler, and with a controlled movement brings the drill pipe from horizontal to a vertical position, and vice-versa when removing pipe.
- For the employment of these rigs under severe climatic conditions, the installation of a disc brake on the drawworks was found more appropriate than the use of a band brake.

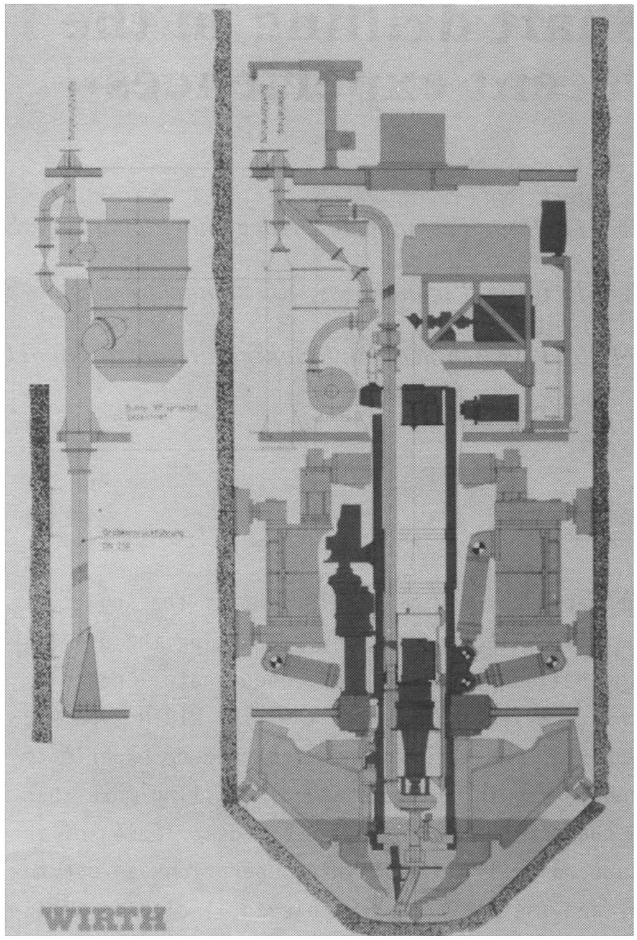


fig. 3: WIRTH fullface shaft boring machine type VSB VI–580/750 (scheme)

- A flowmeter in the return line from the mud pit to the borehole promptly detects possible fluid losses, even in the case of interrupted mud circulation.
- The installation of a metal detector in the circulation discharge end automatically indicates metal particles in the mud. The measuring accuracy being adjusted such that the warning device will respond even in the case of metal the size of the broken tooth of a cutter.
- A drilling data recorder (fig. 8) completes the equipment on the surface, recording continuously the following data:

  - penetration rate
  - hook load
  - torque
  - R.P.M.
  - circulation rate
  - compressor pressure
  - Containerization of the hydraulic equipment of the power pack facilitates maintenance, especially under the climatically unfavourable conditions of the Russian winter.

The capabilities of the rig can be summarized as follows:

<u>Mast</u>

max. exceptional hook load	4250 kN
max. nominal hook load	3335 kN
mast height above ground	29100 mm
max. drill pipe length	12000 mm

fig. 4: WIRTH shaft drilling type L 40

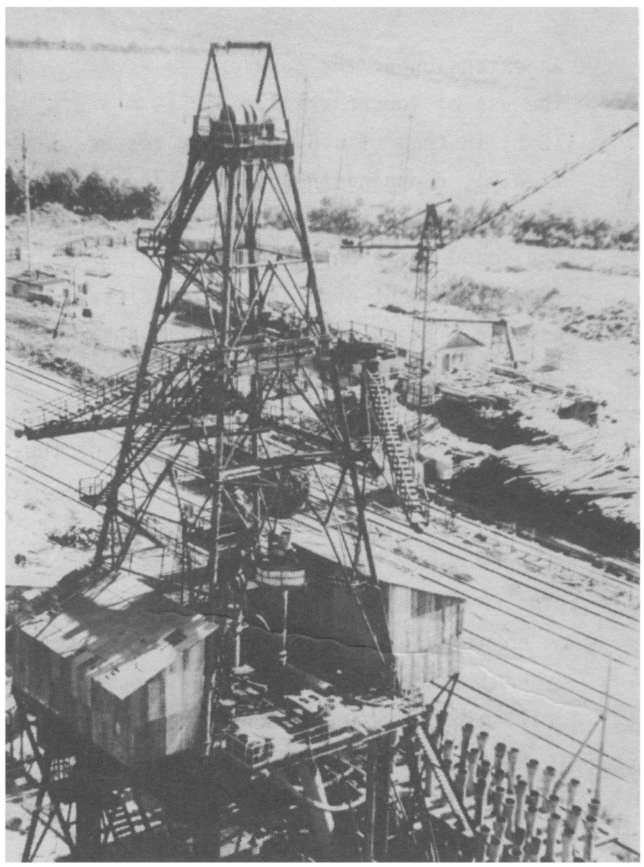


fig. 5: USSR-shaft drilling rig type USTM-6, 2

Substructure
dimensions: 12912 mm×6100 mm, 1–600 sections.

weight	37500 kg
set down load	3400 kN
clear opening (after removal of the rotary table)	5000 mm

Rotary Table

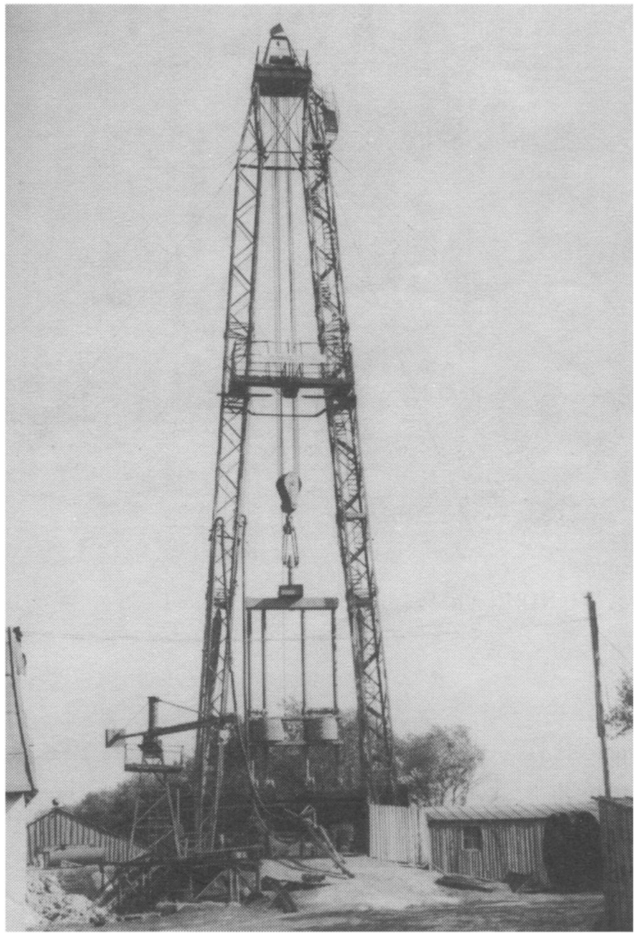


fig.6 USSR-shaft drilling rig type RTB

inner diameter	2110 mm
max. torque	414 kNm
No. of revolutions	0–17 min^{-1}
set down load	3400 kN

<u>Travelling Block</u>	(static load)	4250 kN
<u>Hook</u>	(static load)	4250 kN
Swivel	(static load)	4250 kN

<u>Drawworks,</u> 3-speed shifting gear

max. line-pull, (3rd layer, 1st speed)	365 kN
max. rope speed, (3rd layer, 1st speed)	4.85 m/s
max. hook speed, 12-fold reeving	0.40 m/s

<u>Drill Pipe</u>

weight 330 dia.×6000 mm	1090 kg
weight 330 dia.×12000 mm	1900 kg
torque (at nominal hook load 3335 kN	344 kNm
hook load (at nominal torque 414 kNm)	2450 kN

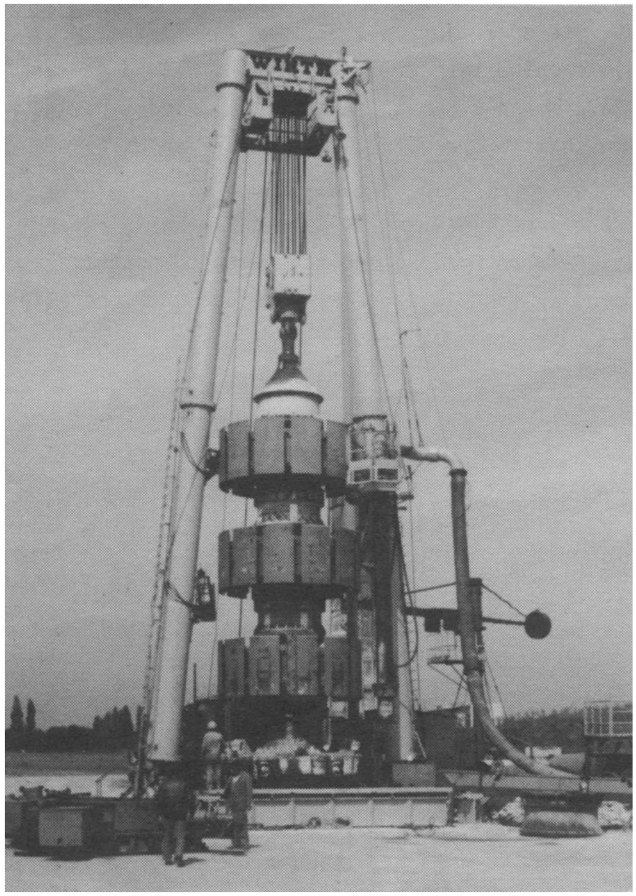


fig. 7: WIRTH shaft drilling rig type L 35

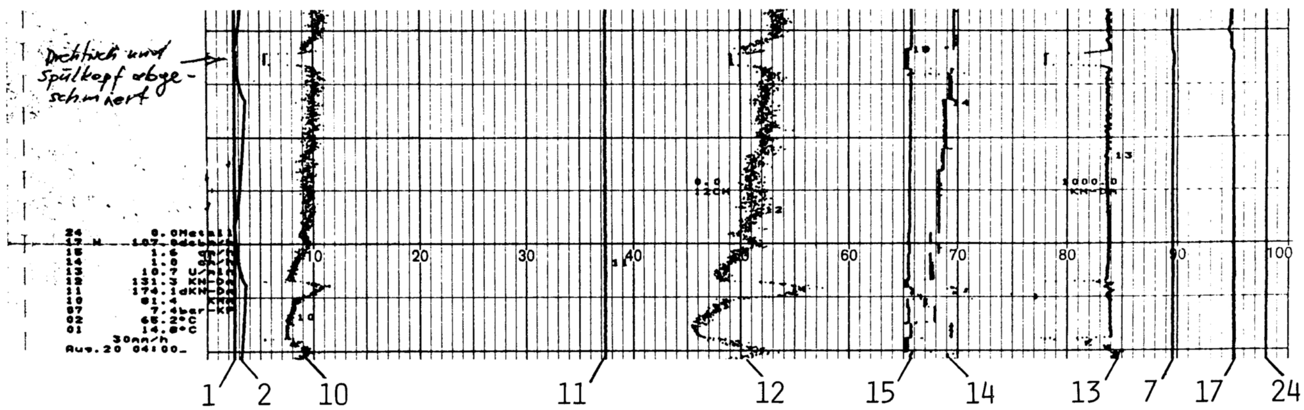
3.2 DRILLING MANAGEMENT

SSB has 4 independent Drilling Administration groups operating in the Ukraine and in the Russian Republic including Siberia (Kuzbas, Krasnojarsk). A plant for the production of the lining tubes and for repair of the entire drilling equipment including the drilling tools is located in Donezk. These five organisations form the association which carries out common planning, research and development, construction and administration, for all drilling sites. In total, 2000 men are employed, 120 of them in the Central Office.

The individual drilling units are delegated by the Drilling Administration to the respective sites which are located up to distances of 5000 km from the base, involving an immense transport expenditure both for material and personnel. For each drilling project, one unit of 25–30 men is used, half of them at a time being employed in shift work on the jobsite and relieved after 4 or 15 days by the other half depending on the distance (fly in—fly out system). During a tour of duty, the personnel stay on the site.

The jobsite personnel consists typically of:

2 engineers (1 chief engineer or mechanical engineer, 1 shift boss)	
8 drillers	
1 crane operator	
1 electro-mechanic	
1 cook	
13 persons	on duty
+ 13 persons	as relief
26 persons	one complete drilling unit



1.	ambient temperature	°C	12.	thrust	kN
2.	oil temperature	°C	13.	rotational speed of rotary table	min^{-1}
7.	compressor pressure	bar	14.	penetration rate	cm/h
10.	torque of rotary table	daNm	15.	max. penetration rate	cm/h
11.	hook load	tons	17.	circulation rate	m^3/h
			24.	metal detector	

fig. 8: printing of drilling data recorder

3.3 PREPARATION

The space requirement of a jobsite is about 100 ×200 m. The location has to be opened up to traffic and be connected to water and electricity supply sources as well as to the telephone network. In some cases (less than 20%), the infrastructure of the contracting mine can be used, but mostly this work has to be planned and executed by the SSB itself.

After removal of the top soil from the jobsite and stock-piling it for the later restoration, the site is levelled and paved with concrete in the area of the drilling rig. Subsequently, the buildings are erected, the settling basin is dug and the shaft feed pipe installed. Simultaneously, all foundations for the drilling unit and the collar shaft are constructed and the mud production and cementing units including silos are also installed All preparation work is executed by a special construction brigade of the respective Drilling Administration.

PREPARATION WORK ON THE DRILLING SITE

Mud Pit:	5 m deep,
	2.5 to 3-fold volume of the shaft to be drilled
Shaft collar:	6 m deep normally
Water supply:	30 m^3/h
Current supply:	6000 V, 800 kW (L-rigs)

After finishing the preparation work, the construction unit is replaced by the drilling unit assembling the drilling rig in 2-shift work and starting it. When the unit is ready for drilling operation, all drilling tools and drill pipe will have been stocked, and auxiliary supplies deposited in silos and stores. In parallel to the assembly, drilling mud is prepared according to the given specification.

Figure 9 shows the typical arrangement of all jobsite equipment for the drilling operation with the L 35 M rigs.

3.4 DRILLING WORK

The collar shaft is drilled by a cutter-head of 4.7 m diameter to max. −12 m depth to enable the complete bottom-hole-assembly to be installed. With this assembly (fig. 10) consisting of cutter-head, skid stabilizers, drill collars and sub to the drill pipe, a conductor is drilled to secure the shaft through unstable strata during drilling operations. Depending on the geology, the conductor length varies from 40 to 120 m. After having drilled the conductor, the complete drill-string is removed, the drilled part is lined by a steel tube of 4.3 m I.D. and backfilled with cement. If unstable overburden layers are not encountered or if they are not thick and are already secured by the collar of the shaft, construction of the conductor is omitted.

During the entire drilling and lining work, the shaft is kept full of drilling fluid to surface.

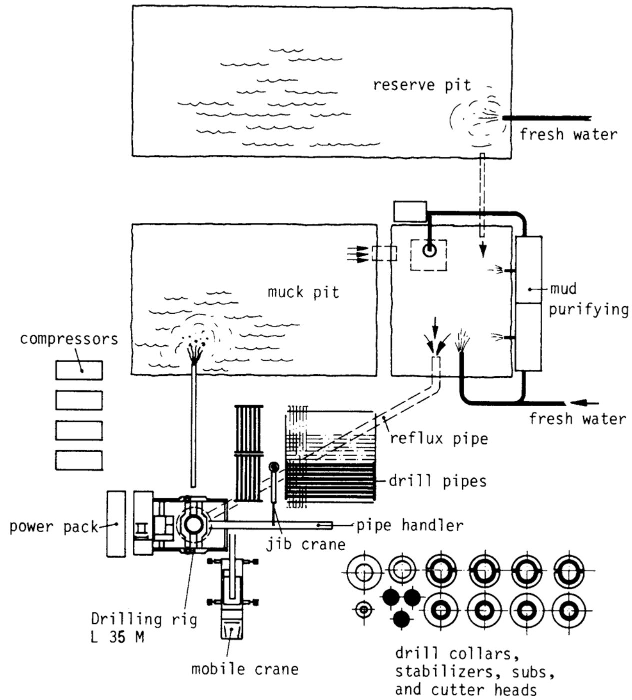


fig. 9: typical arrangement of job site equipment for L 35 M-installations

With an identical bottom-hole-assembly but for a drilling diameter of 4 m, the shaft is drilled from the conductor down to its final depth.

To achieve an optimum penetration rate a drilling program is prepared, depending on the geological formation to be drilled as known from a prior core drilling (strength; inclination of strata; faults; horizons tending to cave or to cause fluid losses). In the programme, are indicated the optimal values for:

– thrust	(hard rock 400–450 kN, soft rock 200–300 kN)
– No. of rev.	(hard rock up to 18 min^{-1}, soft rock 12–19 min^{-1})
– circulation rate	(depending on the operating compressors and on the circulating mixture)
water	= 1300 m^3/h
bentonite	=1000 m^3/h)

To maintain verticality, to the utmost extent, drilling is carried out using l.h. and r.h. rotation, alternating half-hourly.

The air-lift drilling rigs operate with air assisted reverse circulation, the rising speed in the drill pipe reaching up to 5 m/s. The max. dimensions of cuttings can be up to 300 mm.

Drilling is continued up to project depth, which is required shaft depth plus a 15 m shaft sump.

3.5
FLUSHING TECHNOLOGY

The flushing medium essentially has the tasks of discharging the cuttings and supporting the shaft wall. Basically water is used as the flushing medium which, depending on the requirements of drilling technique and the geological conditions, is mixed with polyacrylamide, solely, or with polyacrylamide and some bentonite. Clearwater flushing is used in stable formations, and, when using mine water, detergents are added to reduce water hardness.

In unstable ground, mainly polyacrylamide fluids are used.

In quick sand and loam, flushing with polyacrylamide and bentonite additives is chosen. This type of fluid avoids the dispersion of clay in the water and reduces the filtration properties of the medium. Due to the low specific weight, high drilling speeds are achieved, and the danger of clogging of drilling tools is reduced. Furthermore, flocculation of the cuttings is encouraged which avoids an extended separation time of cuttings in the mud pit.

When traversing varying geological formations, the flushing system is selected on the basis of the most difficult formation as far as drilling is concerned and is normally not changed during drilling.

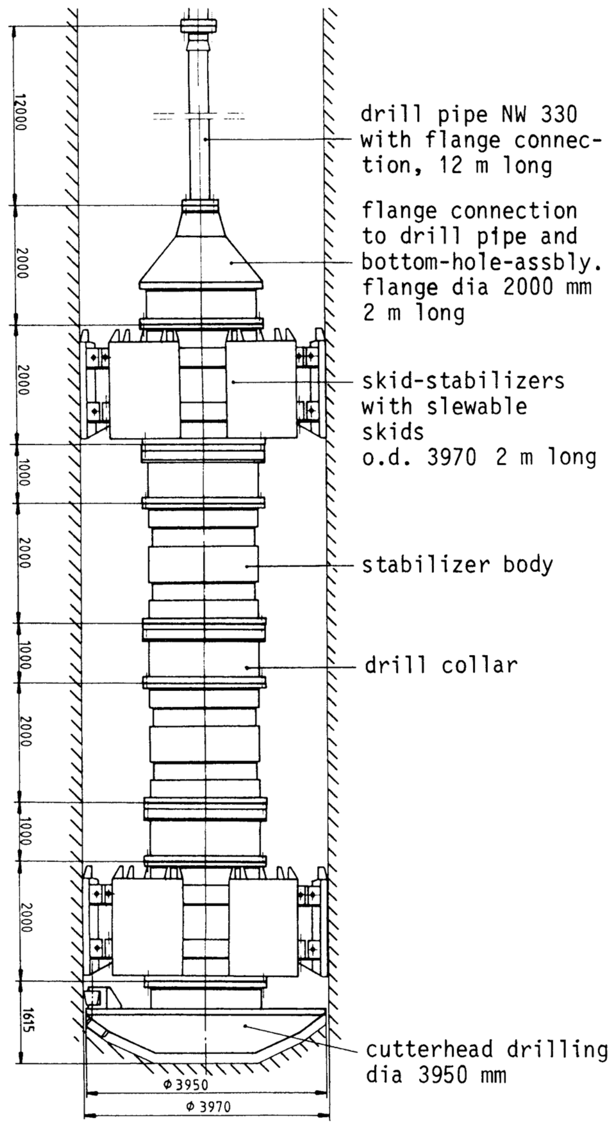


fig. 10: typical bottom-hole-assembly for 3970 mm drilling dia.

The flushing medium is produced either discontinuously by a 2-impeller mixer in a 4 m^3 tank or by a rotating screw with continuous flow. The composition is adapted to the geological formations encountered.

3.6 LINING

Lining is achieved by steel tubes with wall thicknesses of 10–20 mm, depending on the static requirements. Additionally, steel sheet stiffeners 20×300 mm or 160 mm channel profile are weldedon externally at a spacing of 1.0 m to 1.5 m to achieve a higher stiffness. The height of a single lining section is 3 to 6 m.

The individual lining sections are placed on the substructure of the drilling rig, welded to the previous section, and the liner so constructed is lowered by means of a suspension device one length at a time and held by girders for the subsequent installation of the next section. One assembled length should not be longer than 120 m, or as de termined by the diameter and the permissible hook load. After setting the first (deepest) length, by means of the suspension device and utilising the drill pipe, 2 grouting pipes are lowered in the annulus between the wall and lining diametrically opposite to each other. Surface grouting pumps supply enough grout to build up a 2 m high concrete tilting in the annular space and in the shaft interior. For accelerating hardening, CaCl2 is added to the grout. For reasons of safety, a hardening time of 12 hours is allowed. Then the remaining annular space is filled with grout at a rate based on calculations from profile measurements to avoid the lining being crushed by overload. Grout is poured up to approx. 10–15 m below the top of this liner length. By r.h. rotation of the drill pipe, the suspension device is released from the liner length and can be removed together with the drill pipe.

Then, the next length is installed as the first one, but the bottom has a concrete cone which centers itself, when being lowered onto the lining already grouted thus sealing the joint. The grouting rate is dosed as for the first pipe section. Filling is checked every 12 hours and, at the joint, additionally, by grouting pipes equipped with pressure cells indicating contact with the hardened grout. Depth is indicated by the grouting pipe length and the pressure cells indicate the existing grout hardness.

So, the complete shaft is lined by installing a succession of lining lengths and grouting them into position. Thickness of the grout is 200–300 mm, normally.

The grouting pipes are withdrawn in accordance with the pumping rate.

After finishing the lining, the concrete cones are bored out by means of a cutter head the diameter of which is smaller by 200 mm than the steel lining inner diameter, without drill collars and stabilizers; this is done very cautiously to avoid damage to the steel pipes.

3.7
GROUTING

The objective of grouting is to uniformly distribute load created by the rock pressure onto the steel liner, to seal connections between aquifer horizons, and to anchor the lining to the rock. As the steel lining creates a formwork, cavities in the shaft wall arising during drilling operation together with all over size profiles are filled.

Requirements for the grout are:

- pumpability
- hardening underwater
- low segregation characteristics
- good displacement properties for flushing
- lowest possible absorption of mud materials

Grout is used mainly in the mixing ratio: cement: sand: water=1:0.75:0.60.

As activator $CaCl_2$ is added in the ratio: cement: $CaCl_2$=1:0.03 (weight). Hereby, the hardening time is reduced to 4 to 5 hours. For reasons of safety and working organisation, however, each section is left to harden for a period of 12 hours

3.8
DRAINAGE AND CONTROL OF THE SHAFT

For dewatering the shaft, the air-lift system is used up to a residual water level of 70 to 120 m (delivery head limit of the air-lift system), then, underwater pumps in the case of a solids contents up to 0.1% and buckets up to 10 m^3 capacity for solids contents exceeding 0.1 % are used.

Drainage is carried out in sections to be able to continuously monitor the grout backfilling of the steel lining and, simultaneously, to support the uncontrolled section. For control, a two-desk cage with 4 slewable platforms per desk is utilized permitting personnel to work at the shaft lining. This cage is handled by means of a rope from a separate winch over a pulley in the drilling mast. Additionally, a second winch is installed for an emergency cage; this winch is driven electrically or manually. During the control work, the shaft is ventilated by steel ducts of 180 to 200 mm diameter at a ventilation pressure of 1.8 bar.

When cavities are detected during the grouting, they are backfilled by injection. For this purpose, pipe plugs are installed in the steel lining which can be opened for control and can be used to connect the injection pipes to the lining. Damage to the metal lining is repaired by welding.

After complete pumping out, the shaft is surveyed and handed over to the client. Until the shaft is connected to the underground workings of the mine maintaining of a minimum water level may be necessary to compensate for pressure fluctuations caused by the driving of the connecting roadway.

3.9
DEMOBILISATION AND RESTORATION

The drilling Unit disassembles the drilling rig and all other work is done by the construction unit. In addition to the disassembly of all jobsite facilities, the mud pit has to be filled and the top soil has to be spread again.

4.
CONSTRUCTION PERFORMANCE

An average analysis of the time spent in the construction of drilled shafts of 500 m depth and 3.5 m inner diameter was,

jobsite preparation	4 months	=	19%
drilling operation	12 months	=	57%
lining and grouting	3 months	=	14%
drainage and control	1 month	=	5%
restoration	1 months	=	5%
	21 months	=	100%

For the shaft with the shortest construction time Nowogrodowskaya, rig No. 1 L 35 M 570 m deep; 3.5 m I.D. the analysis was:

jobsite preparation	2 months	=	11%
drilling operation	11 months	=	61%
lining and grouting	3.5 months	=	19%
drainage and control	1 month	=	6%
restoration	0.5 month	=	3%
	18 months		100%

Performance achieved

	average	max.
drilling lining and grouting finished shaft (without preparation and restoration)	41.7 m/month	123 m/month
170 m/month 31.5 m/month	200 m/month 38 m/month	

5.
PROBLEMS ENCOUNTERED

During the construction of 25 shafts finished so far numerous recurring problems were encountered, which were all in connection with the directional accuracy and the penetration rate. An exception to these are the ‘fishing operations’ due to causes ranging from the occasional dropping of a tool into the shaft through losing of individual cutters to the loss of the complete bottom hole assembly.

5.1
FISHING

As an example of how extensive such fishing operations can be, an incident which occurred in October 1986 at the Butowka shaft is described below.

After drilling through a sandstone formation of nearly 35 m, a clay schist zone was encountered at depth −276, 2 m. At depth −284, 7 m drilling was interruped by an electrical breakdown of 2 hrs. Starting drilling again was impossible due to the sticking bottom-hole-assembly. Applying 350 tons lifting force to the drill string allowed the movement of the bottom-hole-assembly to depth − 242,5 m which is above the beforementioned sand-stone formation. During this period the shaft was filled with bentonite mud with a specific gravity of 1,3 g/cm^3 due dissoluted mud out of the rock strata.

It was decided to reduce the filter cake alongside the sandstone formation by reaming. This operation started Oct. 8, 1986, and was commenced next day when the cutterhead had reached −284,2 m again and the mud supply tube to the shaft was blocked and circulation interruped. After 2 hrs of cleaning the tube circulation was in operation again. However, continuation of drilling was impossible due to the again sticking bottom-hole-assembly (fig. 11).

After 4 days of constantly repeated pulling with 340 tons and 15 rpm and loading with 140 tons without rotating the cutterhead no success could be achied in getting the sticking parts loose.

Supposing the sticking of the upper stabilizer plates in the sandstone formation caused by newly built-up filter cake, it was decided to inject 300 m^3 of clear water in order to reduce the filter cake by dissolution. Therefore, a 60 mm dia pipe was

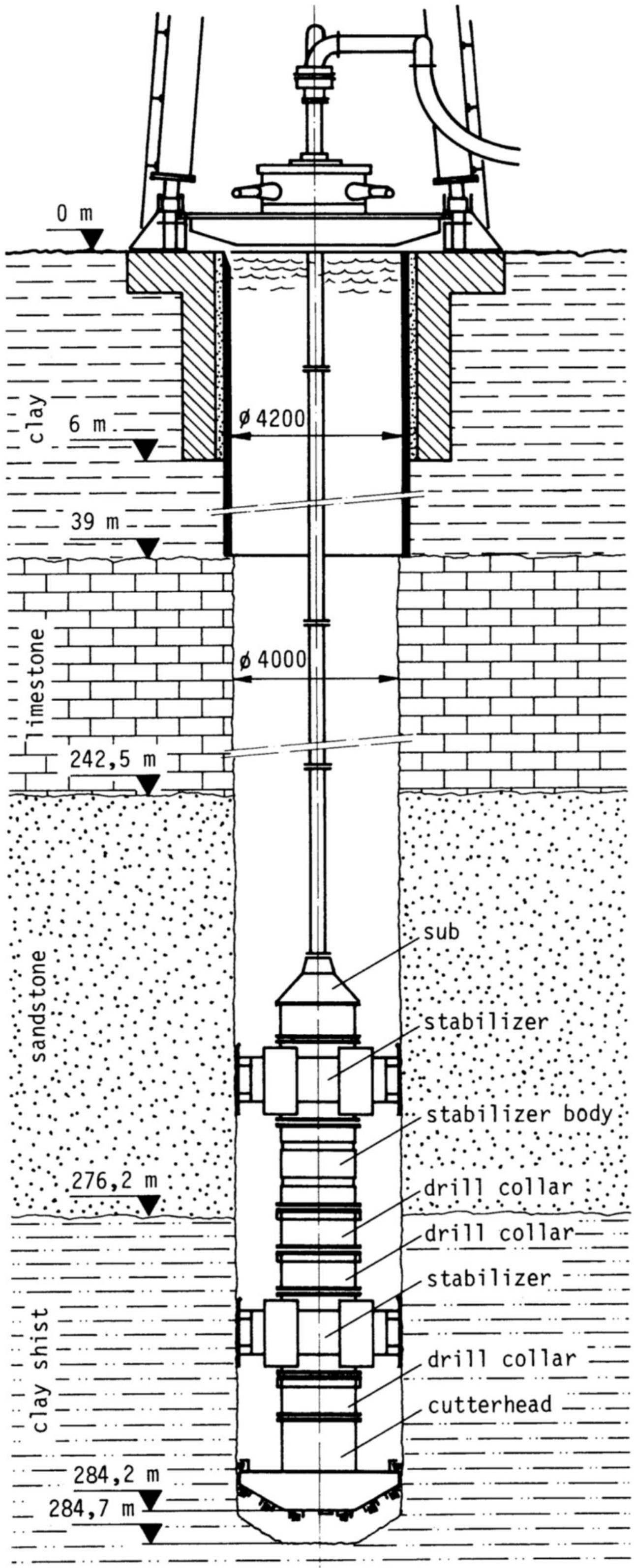


fig. 11: position of bottom-hole-assembly after sticking in Butowka shaft

lowered to −283,2 m and the water was pumped into the shaft underneath the two stibilizers (fig. 12). For 3 days rotating the cutterhead with 10–15 rpm in order to produce vibration and pulling with up to 340 tons was unseccessful but the stabilizer bearing broke resulting in a blockage of the cutterhead rotation which could not be overcome even applying the max. torque

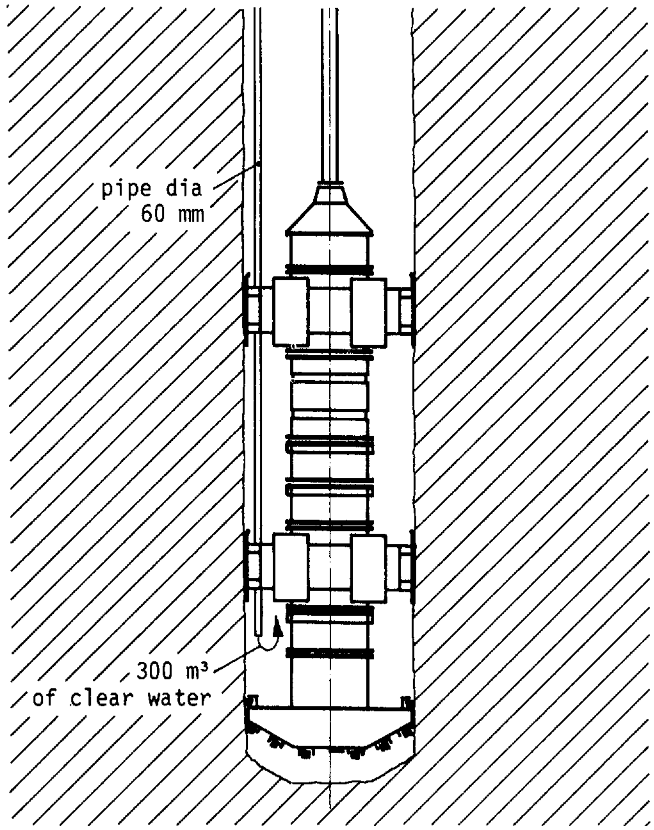


fig. 12: clear water injection for the dissolution of filter cake

of 414 kNm of the rotary table to the drill string. The pulling force was furthermore increased to the max. exeptional hook load of 4200 kN for approx. 2 hrs. No movement of the bottom-hole-assembly had been achieved.

In order to assist the lifting of the sticking drill string it was decided to use explosives to be detonated underneath the cutterhead. Therefore, the elbow-tube of the swivel was opened to allow the passing of a 500 mm long 300 mm dia explosive container into the drill pipe to be lowered by the ignition cable. Preparation of the swivel, manufacturing of the blasting container, and getting permission for blasting lasted 12 days.

Oct. 31, 1986, first blasting was carried out. The charge of 10 kg of explosives was lowered and, after applying of 340 tons pulling force to the drill string, detonated with the result that the pulling force was reduced to 310 tons and a mowing of the drillstring of 50 mm upwards.

After checking of the clearance inside the drill pipe the second charge of 10,5 kg was lowered and fired while the drill pipe was under pulling force of 340 tons which reduced to 325 tons after detonation, the lifting of the drill string amounted to 30 mm.

Repeating the procedure described above the third charge of 15,5 kg was detonated resulting in a remaining pulling force of 310 tons and a lifting of 35 mm.

Checking the pipe clearance after the third explosion showed a damage at −273,6 m which blocked the passing of the explosive container. To continue blasting assisted lifting it was decided to detonate the explosives in the area of the upper stabilizer close to the shaft wall in order to get the stabilizer skids out of their sticking position. Therefore, 150 mm wide and 1,5 m long explosive containers have been fabricated.

Nov. 2, 1986, the fourth charge of 6 kg was lowered and fired after applying of 340 tons pulling force to the drill string, which reduced to 310 tons after detonation.

The fifth charge of 6,8 kg caused a reduction of the applied pulling force from 380 tons to 355 tons. It is supposed that this charge detonated at the transition of the conductor to the shaft at − 39 m only causing serious damages to the lining.

The sixth charge of 7 kg reduced the applied pulling force from 380 tons to 335 tons. The seventh charge of 8 kg was fired on Nov. 4, 1989. The applied pulling force was reduced from 335 tons to 50 tons and drill string could be rotated again. The supposition that the drill string was broken had been verified by pulling out the drill string including the cross-over sub without lower connecting flange (fig. 13).

During the period from Nov. 5 to Nov. 30, 1986, the actual position and the attitude of the remaining parts in the shaft had been investigated by using several blank and conical moulds. Analysing the impressions into the moulds it was detected that

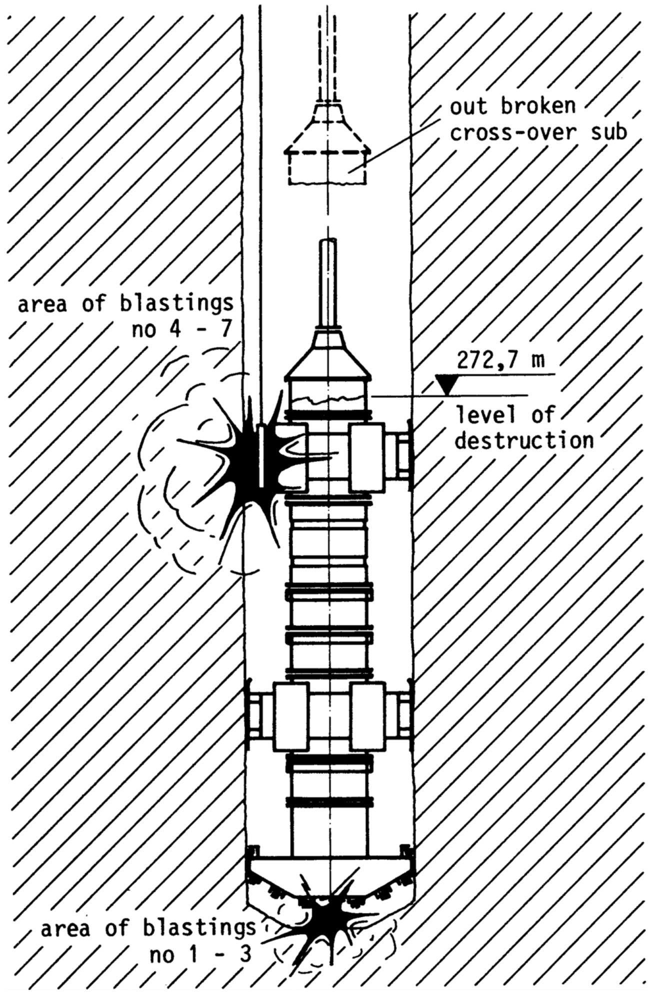


fig. 13: blasting operations to assist the hoisting of the bottom hole assembly

- the upper end of the bottom-hole-assembly was the missing flange of the cross-over sub;
- the flange had a central hole of 350 mm dia;
- the flange was not covered by material out of the shaft wall;
- the flange was tilted by 4 degrees measured to the horizontal.

Based on these facts the concept was chosen to enlage the shaft above the bottom-hole-assembly to 4300 mm by means of an unballanced cutterhead in order to allow the installation of a wash-over tool to drill behind the sticking stabilizers plates. Until Nov. 30, 1986, an existing 4 m dia cutterhead was modified by applying a balance-error load of 7 tons onto the cutterhead and at the opposite side the necessary reamer cutters. Also, the wash-over tool was manufactured.

Starting Dec. 1, 1986, the unballanced cutterhead reamed the shaft from –268 m to –272,7 m to 4300 mm in dia, finishing this operation on Dec. 16, 1986 (fig. 14).

The wash-over tool came into operation the following day; using a spear as guidance inside the 350 mm dia central hole of the bottom-hole-assembly, the stabilizer skids had been washed over from −272 m to −274,15 m lasting 29 days (fig. 15).

Jan. 15, 1987, the bottom-hole-assembly was fished by a spear in the inner pipe. The load indicated that only the upper stabilizer had been fished, the third explosive detonation must have destroyed the connection of the bottom hole assemply underneath the upper stabilizer. Unfortunately, when hoisting the upper stabilizer to surface its tilted attitude caused it to jam in the shaft at −264,6 m depth. Pulling with 350 tons was unsuccessful, therefore, the fishing tool was disconnected from the stabilizer and hoisted to surface.

The impression on a blank mould showed that the stabilizer was sticking in a tilted position of 9 degrees (fig. 16). Applying a load of 120 tons to the stabilizer by means of drill-collars in order to press the stabilizers downwards had no positive result and was stopped Jan. 23, 1987. It was decided to change the circulation mud with new bentonite mud with polyacrylamid additives and subsequent enlarging of the shaft dia to 4130 mm. From Jan. 24 to Feb. 13 the new mud of 1,03 g/cm^3 was

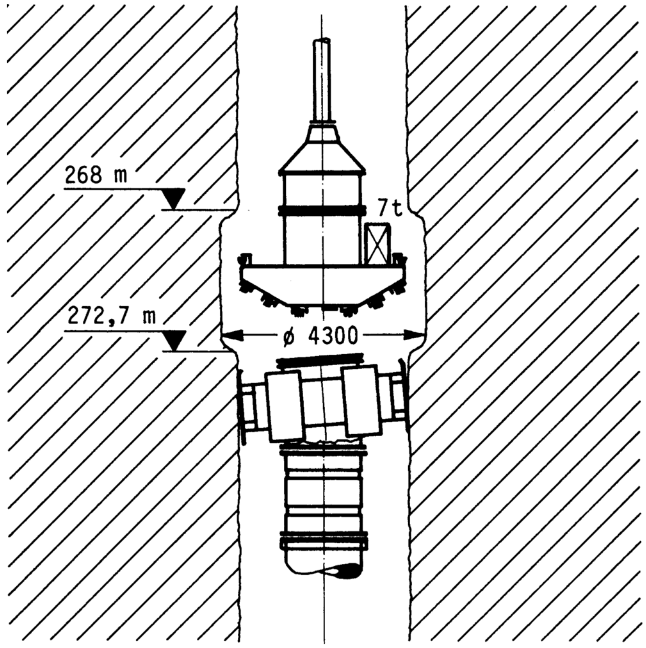


fig. 14: enlarging the borehole diameter by an unballanced cutterhead

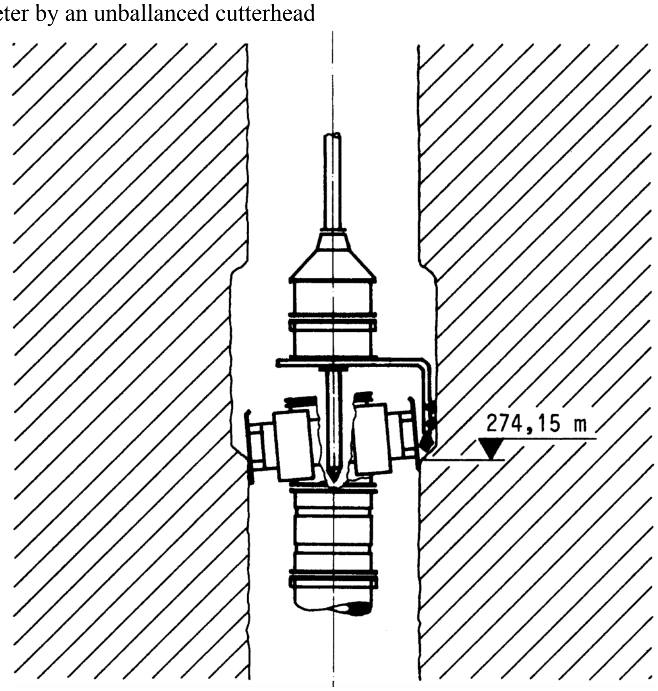


fig. 15: wash-over tool reaming behind the stabilizer skids

flowing into the shaft from surface while replaced mud was air-lifted from −264 m. For the following 20 days the shaft was reamed to 4130 mm from −39 m to −261,6 m depth.

March 6, 1987, the stabilizer was washed over again from −263,1 m to −265,6 m until March 27, 1987. The next day the stabilizer was fished and hoisted to surface on March 29, its position leaving the shaft was tilted by 4 degrees.

By different moulds it was detected that the remaining parts in the shaft were topped by a tube of 320 mm dia and 5 mm wall thickness at −272,2 m and testing the depth of the shaft cross-section by using the drill pipes showed that the area above the lower stabilizer was filled up to −274,15 m by cuttings produced during the reaming operations.

It was decided to wash-over the drill-collar section down to −278 m which was carried out using a wash-over tool of 3200 i.d. from April 7 to April 17, 1987 (fig. 17, 18).

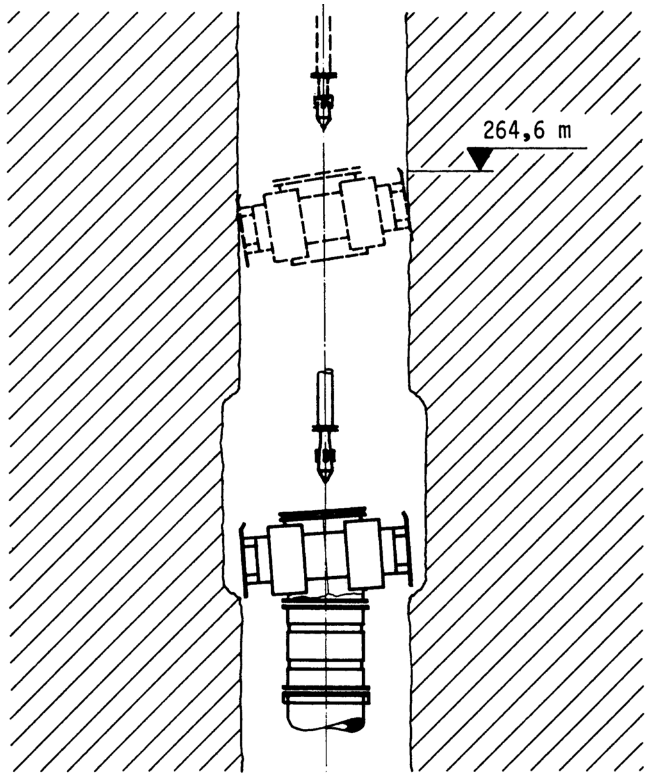


fig. 16: fishing operation of the upper stabilizer

Subsequently the bottom-hole-assembly was fished and hoisted to −39 m where the assembly was blocked and after repeatedly pulling the fishing tool lost the connection followed by the dropping of the bottom-hole-assembly on April 21 (fig. 19).

Based on the fact that the fished bottom-hole-assembly could not pass into the conductor section, it was obviously that the lower end of the conductor lining must be destroyed and did not allow free passage of the stabilizer and cutter-head. Therefore, the shaft was sumped to −40 m, an operating platform was installed and lowered by a separate winch to repair the destroyed conductor lining which has left a free passage of 3800 mm only. After the repair the shaft was filled up to surface with mud again. This operation lasted 13 days.

By means of blank moulds it was detected that the dropped head stood at the shaft bottom tilted by 18 degrees measured to the vertical, the tube on top 700 mm aside from the shaft wall. Using an articulated fishing tool the bottom-hole-assembly was hoisted but the cutterhead itself without connecting flange remained at the shaft bottom; this operation including manufacturing of the articulated fishing tool lasted 14 days and was commenced May 17, 1987 (fig. 20).

The wash-over tool which had been used before was then installed to wash over the upper area of the cutterhead to allow exact determination of the attitude of the top side of the cutterhead. The blank mould impression showed the deformation and also ribs. With a catching device using slings an attempt was made to draw the cutterhead, however, it dropped after 4 m of hoisting again.

Using a 1500 m long steel bar, horizontallly mounted underneath the drill pipe, the configuration of destruction was detected.

Another attempt with the sling catching device failed after 47 m of hoisting.

Another mould impression taken showed additional damages on the top side of the cutterhead which did not allow further attempts with sling-type catching devices. A circular catching device was manufactured (with 3 self-acting latches) and three attempts of fishing the cutterhead failed when the connection got loose after applying of 20 tons pulling force only (fig. 21)

It was decided to mill the centre pipe of the cutterhead used for circulation from 330 mm to 550 mm to achieve as much surface as possible to use the modified fishing tool operating as an internal gripper (fig. 22). With its first attempt the cutterhead was hoisted to the surface. The serious damages did not allow repairing for further employments. Smaller steel parts had been taken out of the shaft by means of a grab.

On June 25, 1987, 8 month and 17 days after sticking of the bottom-hole-assembly, the fishing was terminated.

This example taken from practical operations is —with regard to the time requirement—not typical; fishing of bottom hole assemblies, as are necessary occasionally, normally require 36 to 48 hours. A typical example is a drill pipe breakage, as occurred with the L 35 rig No. 2 at job site Abakumova in Nov. 87 at a depth of −327 m.

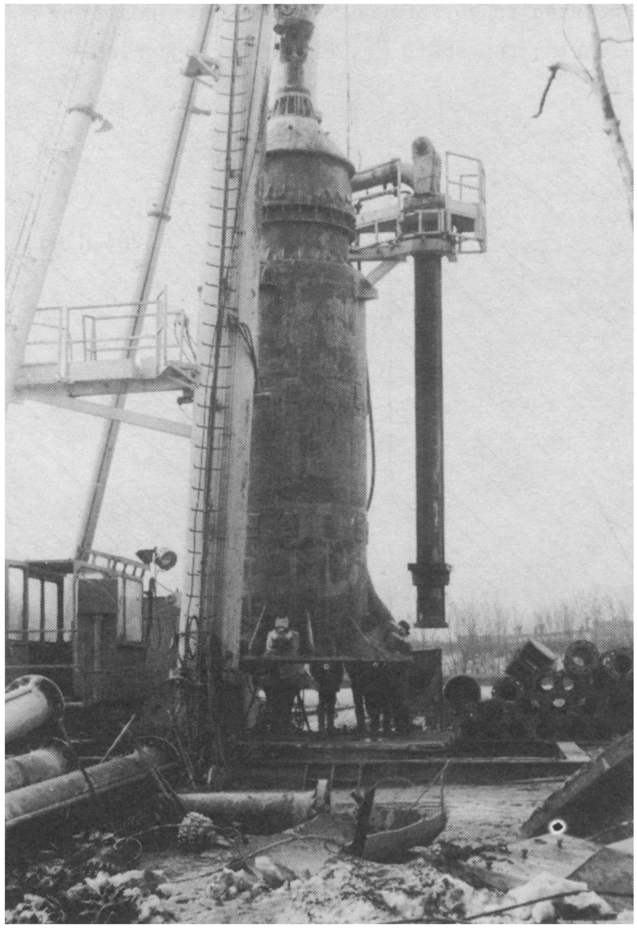


fig. 17: bell-type wash-over tool ready for lowering

By means of a catcher, fixed to the drill string, within a centering device, the total bottom hole assembly (fig. 23) was fished at the break in the first drill pipe section above the cross-over sub at the first attempt. The total operation took only 2 shifts.

Excluding the sometimes long periods for fishing, it can be said that the economy of shaft drilling in the Donezk hard coal area is essentially influenced by the overall drilling rate which is governed by the necessity to maintain verticality.

5.2 PENETRATION AND DEVIATION

It is well known that high penetration rates and minimum deviations are very seldom achived at the same time. This is valid, as long as drilling relies upon the pendulum effect, which is minimized when the majority of the weight of the bottom hole assembly is used for loading the cutter head. The first type L 35 rigs used drilled with a bottom hole assembly consisting of

1 drill head
2 stabilizers
3 drill collars
1 sub

resulting in a length of 8,6 m measured from the bottom of cutter head to the upper flange of the top stabilizer. The results achieved, which were clearly not satisfactory, made it necessary to change the composition of the bottom hole assembly to:

1 cutter head
4 stabilizers
3 drill collars
1 sub

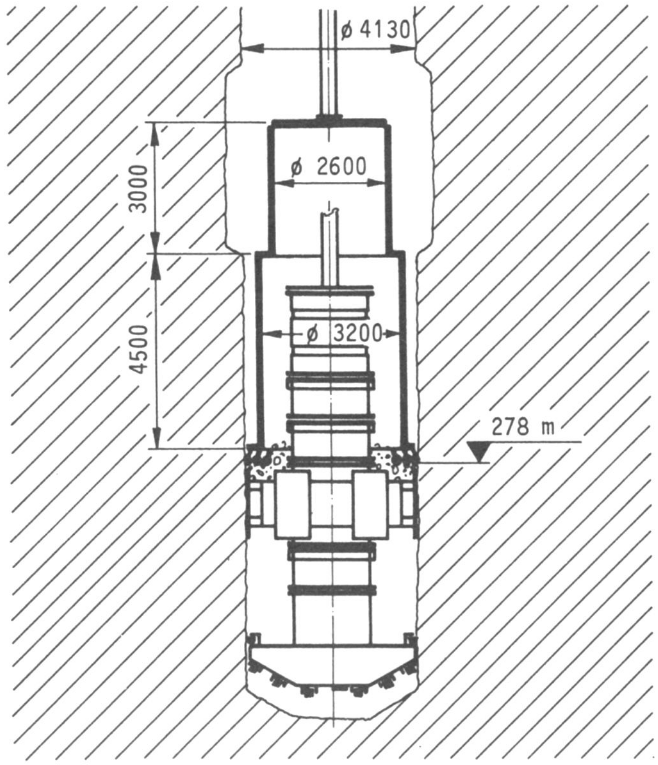


fig. 18: bell-type wash-over tool (schem of operation)

with a length of 12,6 m measured as before.

Experience with 4 stabilizers however showed that 2 stabilizers arranged at a spacing of 9 m maintained direction to the same extent as 4 stabilizers were arranged directly one above the other. Nevertheless, such a bottom hole assembly make-up does not guarantee drilling a vertical shaft under all geological conditions.

When there are significant deviations, exceeding the admissible 1:1000, the shaft must be totally or partially reamed. For this purpose, the drill head is drawn up and slowly lowered with maximum rotary speed to the bottom. Due to the pendulum effect, the drill head has a tendency to cut into the low-lying side of the shaft wall thus enlarging the shaft. This is continued until through the shaft enlargement, the deviation from the shaft axis is acceptable (i.e. within the tolerance given).

With the geological conditions in the Donezk area, 10–25 % of the total drilling time is required for reaming, in individual cases however it can be more than 50%. Therefore, it was decided, in the case of especially unfavourable geological conditions, to drill with a bottom hole assembly of the following make-up

1 cutter head
2 stabilizers
3 shaft collars
1 extension piece (stabilizer body)
1 sub

The distance from the bottom of the cutter head to the uppermost stabilizer in this case amounts to 10,6 m. With this assembly, potential enlargement of the diameter created by the eratic motions of the cutter head in all directions is deliberately accepted. This causes a considerably increasing material requirements, but reduces the corrective reaming time to an extent, which economically can be accepted.

The stability of direction can also be improved by adapting the drill head shape to the rock conditions. In the case of the type L rigs, cutter head shapes from a slightly conical shape through an absolutely flat cutter head to a concave cutter head shape (the so-called W-head) were tested (fig. 24). The concave shape shows a considerably better stabilizing tendency than the conical cutterhead, whereas the use of the flat cutter head shows no advantages. However, it must be pointed out that the W-shaped cutter head is the most unfavourable shape with regard to cleaning the hole bottom of muck.

The removal of muck was another problem, which was approached step by step in the course of drilling the 25 shafts completed up to now, as far as increase of the penetration rate was concerned.

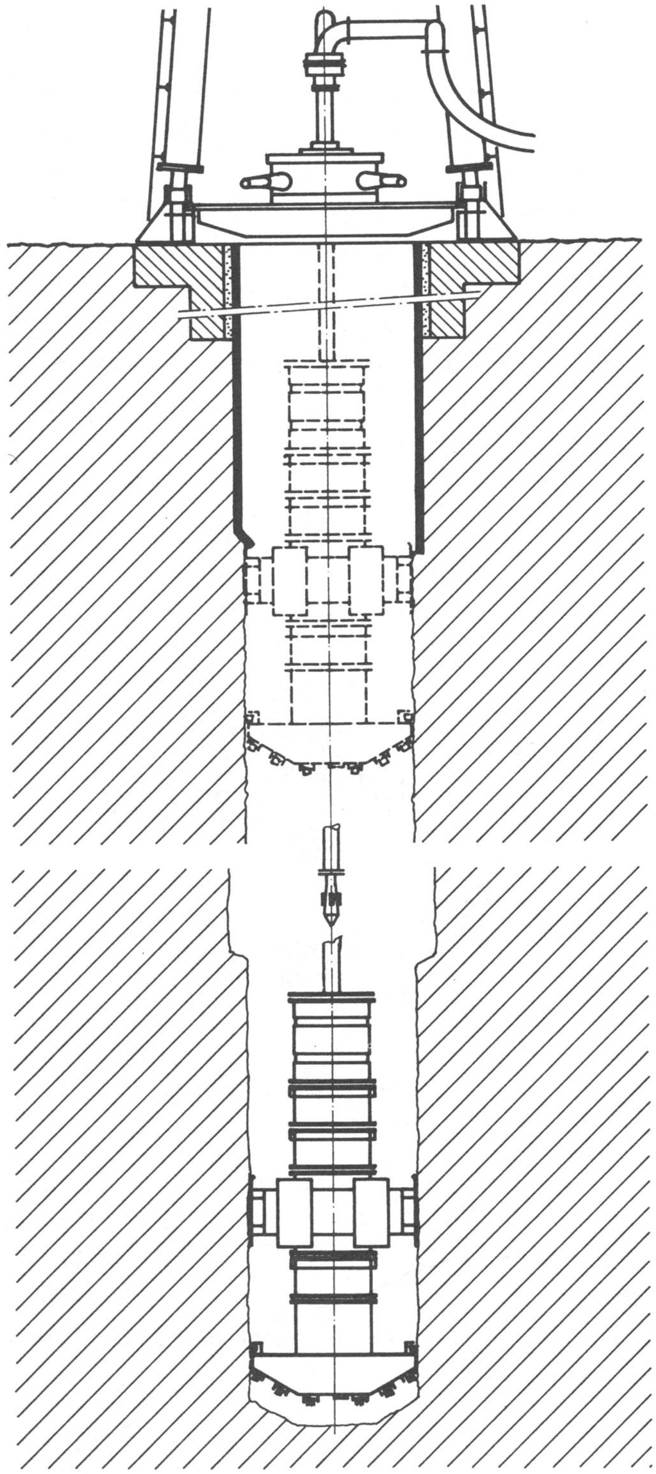


fig. 19: the fished bottom-hole assembly blocked by the conductor lining

For improvement of the hole bottom cleaning, especially in sticky clay formations, which are encountered frequently, the number of cutters was reduced (from 25 at 4 m of diameter to 14). However, this was only possible by the development of toothed cutters, which work on the shaft bottom with their complete width, so that a pair of cutters per track was not necessary.

The position of the pick-up suction pipe was checked as well. It was found out that an eccentric position near the centre is the most favourable position. Subsequently, the flow patterns in the space between the shaft bottom and the cutter head face were improved in such by providing the cutter head with a skirt at the circumference. This skirt is only broken at the positions of the gauge cutters and extends to within 70 mm of the shaft bottom.

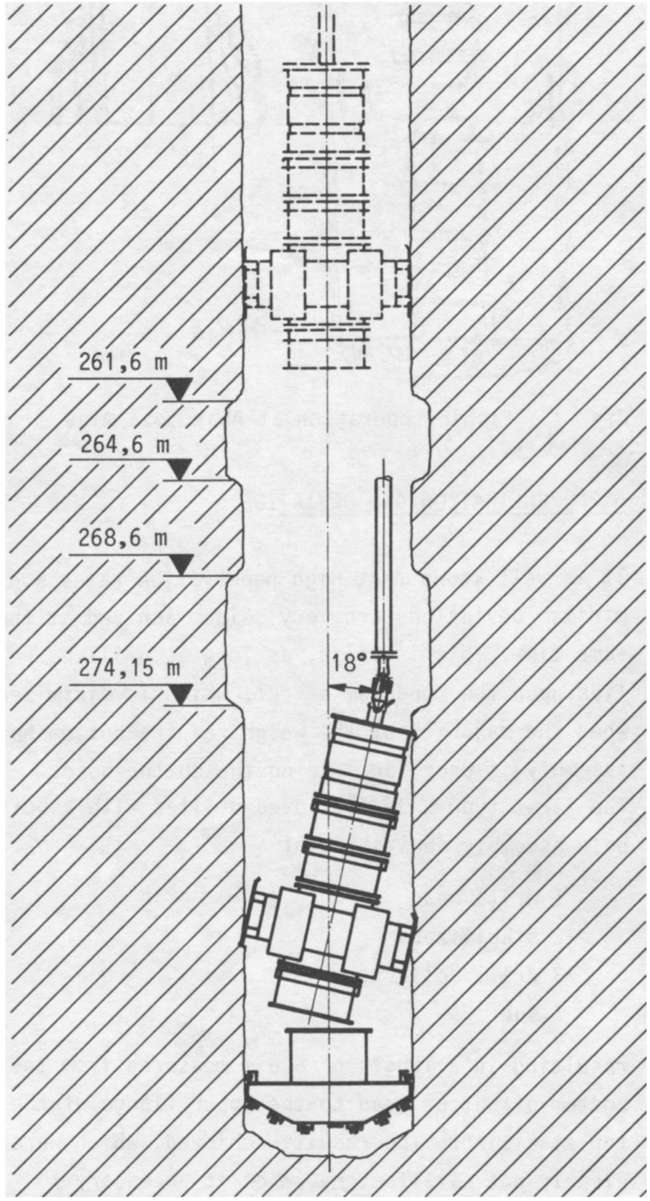


fig. 20: the fishing of the dropped bottom-holeassembly

On the cutter head itself, additional baffle plates were installed, which led the flushing flow nearer to the shaft bottom. For reasons of completeness, it should be mentioned that the mounts for the cutters were modified in such a way that both r.h. and l.h.turning of the drill head is possible without risking the security of the cutters fastenings.

However, problems of reconciling maintenance of direction and increasing the penetration rate still existed. Therefore, SSB and WIRTH jointly decided to develop a steerable drilling device (SDD).

6.
THE STEERABLE DRILLING DEVICE

The basic requirements for this Steerable Drilling Device (SDD) were defined as follows:

1. The method must be suitable for at least a depth of 1000 m (3280').
2. The method must permit the construction of shafts of a cross section of 80 m^2 (862 sq feet).
3. The method should be suitable for unstable, for aquiferous and also for dry rock formations.
4. The gross drilling rate must correspond to at least the average performance of the conventional sinking method.
5. The shaft must be within acceptable tolerances of the designed axis.

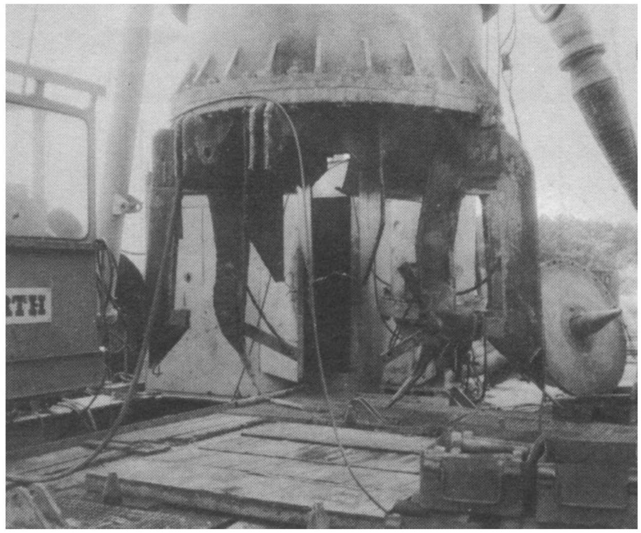


fig. 21: the circular catching device with three self-acting latches

Within the scope of the development work, the different mechanical construction possibilities were discussed and checked first. The most essential distinguishing characteristic of the different variants was the drive system for the steerable drilling device. As alternatives, the following possibilities were considered:

- rotary drive by the drilling rig installed at the surface, i. e. with rotating drill pipe,
- underwater drive for the steerable drilling device, i. e. permitting stationary drill pipe.

Due to the better possibilities of realising construction with respect to measuring technique, data transmission and power transfer, the variant with underwater drive was chosen for the prototype.

In the case of this variant, the stationary drill pipe functions only as a mud channel, a torque compensator and a handling device for the steerable drilling device.

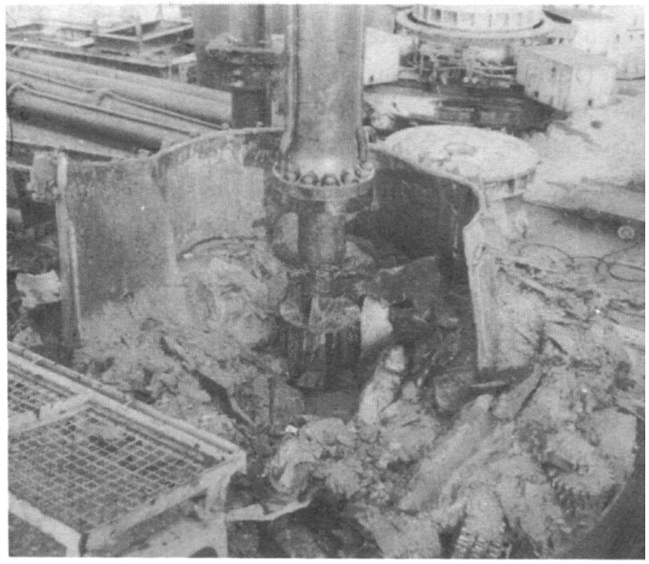
For the prototype, a cable, clamped to the drillpipe sections, is used for data transmission.

Essentially, this steerable drilling device comprises the following main components (fig. 25):

1. Cutter head
2. Lower cutter head bearing (main bearing)
3. Nonrotating guide weights
4. Guide frame clamped hydraulically to the borehole wall at two levels
5. Electric drive and gearing for rotating the cutter head
6. Hydraulic power pack
7. Electronic inclinometers
8. Power supply and data cable, attached to the drill string
9. Console with all instruments and controls
10. Electrical switch gear enclosure.

The guide frame with the clamping and steering pads forms an outer kelly, the guide weights, main bearing, drive, as well as the hydraulic tank and the electric enclosure together form an inner kelly, analogous to TBM construction.

The cutter head is rotated by a pipe, by means of an electric drive and gears, the so called power swivel, incorporated in the steerable drilling device. This pipe transmitting the torque, is connected by means of a flange to the rotating part of the main bearing. The cutter head is bolted by a flanged connection to the main bearing. Diaphragm accumulators continuously compensate between the depth-dependent outer pressure, and the correspondingly sealed inner pressure of the main bearing, and the drive bearing. The drilling thrust is generated by the nonrotating guide weights, which can slide downward within the guide frame, acting directly onto the main bearing. This offers the advantage, that the nonrotating loading weights dampen the reaction forces resulting from the drilling operation, so that the shaft walls are less stressed than they would be if using conventional bottom hole assemblies with sliding stabilizer skids and/or rotating drill collars.



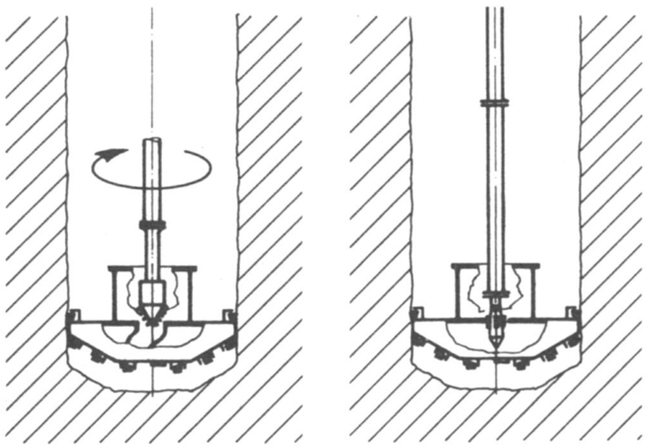


fig. 22: milling of centre hole and fishing of the cutterhead, view at the hoisted cutter-head leaving the shaft

The specific cutter head load is adjusted to suit the geological formation by the feeding rate, which is controlled by the feeding device of the drilling rig on the surface. Using the steerable drilling device, almost the complete equipment weight except that of the clamped guide frame can be used as cutter head load.

The device drills in strokes equal to that of the inner kelly. Prior to starting a stroke, the freely suspended cutter head is aligned vertically, or at the inclination and direction desired for achieving a correction of direction by means of the upper steering cylinders—individually controllable— actuating the clamping pads (fig. 26).

After drilling a stroke, the cutter head is stopped, the clamping system of the guide frame is retracted and, then the whole steerable drilling device is slightly lifted. Subsequently, via reset cylinders, the guide frame moves again into the starting position, ready for the next stroke.

Hydraulic pump, drive motor and valves are arranged in a closed oil tank on the inner kelly. Diaphragms act for pressure compensation on the oil tank with in-creasing depths and as volume compensation when actuating the differential cylinders.

In case of failure of the hydraulic power pack (e. g. cable breakage), an emergency system ensures that the steering and clamping cylinders are automatically retracted, so that the steerable drilling device can be pulled out.

The cuttings are removed by means of the airlift method. The compressed-air inlet nozzle being arranged directly above the power swivel.

The drill string does not rotate and just serves as a mud channel, torque compensator, for transmitting the pulling force, and for running in and pulling out the steerable drilling device.

Directly above the drilling device, there is a protection against caving ground falling between the wall of the drill hole and the bit body.

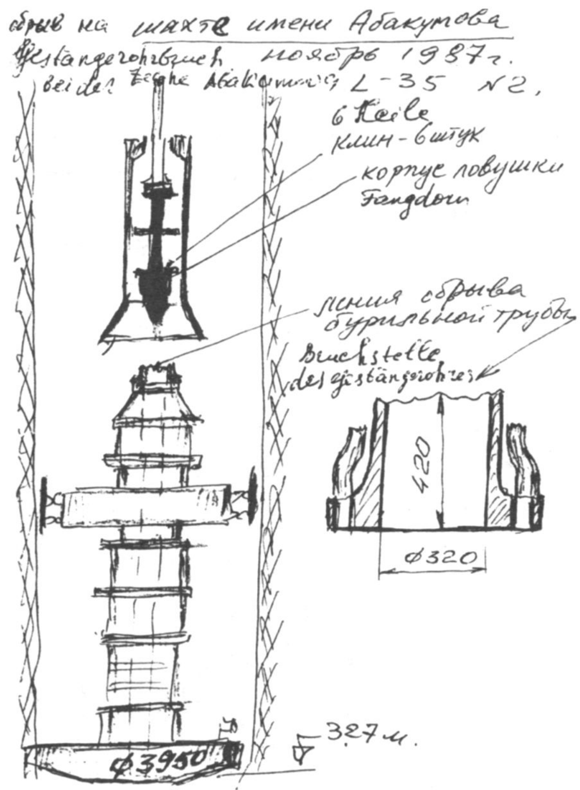


fig. 23: fishing operation at Abakumova mine

6.1
FIRST TRIAL

The Rheinische Braunkohlenwerke kindly declared itself prepared to test the prototype of the steerable drilling device within the scope of a water well drilled for the open-cast mine “Hambacher Forst” (fig. 27). For this test, the “BOWA” (Drilling and dewatering dept. of Rhein. Braunkohlenwerke) used an air-lift drilling rig WIRTH L 15 S with a rotary table of 2110 mm (83") I.D. The formations to be drilled consisted of sand, gravel and clay.

The length of the standpipe was 37.50 m. The inner diameter of the standpipe was 2210 mm (87").

Due to an inclined position of the standpipe and difficulties of adaptation in handling the steerable drilling device, there was at a depth of approx. 45 m (148'), i. e. the depth at which the steerable drilling device had entirely left the standpipe, a deviation of approx. +160 mm (6,30") in the+y direction and approx. 50 mm (1.97") in +x. The direction +y was in this case defined towards the drawworks of the drilling unit.

The course of the drill hole can be seen from fig. 28. The initial deviation was corrected very quickly. In order to investigate the steering qualities of the steerable drilling device, a deviation was purposely produced within the range of 70 m to 90 m (230'–295') depth, and subsequently corrected again.

It was intended to carry out a routine examination of the steerable drilling device after about 100 m (328') of drilling. In practise this was carried out at a depth of 147.4 m (483,6'), i. e. after 109 m (357,62') of drilling. During this inspection of the steerable drilling device on surface, the drill hole was filled with mortar between the depths of −147 m to −138 m (−482' to −453'). The compressive strength of this after 24 hours was more than 20 N/mm^2 (2900 PSI). Collaring in the mortar plug was effected without any difficulties. A short-term deviation in the −y direction was caused by the sluggish motion of one clamping cylinder. After turning the steerable drilling device 180 degrees, the deviation was corrected immediately.

On the first section up to 147.4 m (483,6') depth, the average penetration rate was approx. 2.01 m/h (6,6'/h), including all deadtime for steering, resetting, adding drill-pipe section, and for consideration of the next steering operations. On the second section from 138.4 m to 201.4 m (454,07' to 660,76'), the average penetration rate was 1.94 m/h (6,37'/h); including all a/m deadtimes. The average net penetration rate, i.e. without the deadtimes, amounted to 4.34 m/h (14,24'/h). For this section, the following time utilization analysis can be indicated:

– Drilling 44 %
– Adding drill-pipe section including measuring
25%

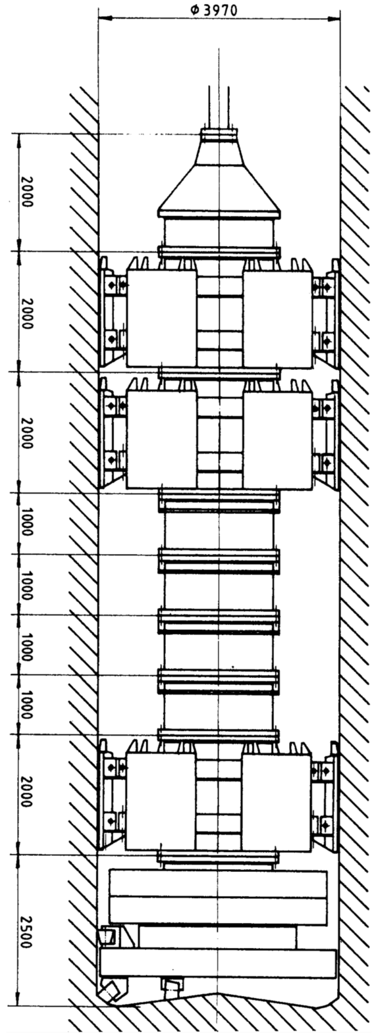


fig. 24: WIRTH bottom-hole-assembly with w-shaped cutterhead

– Resetting of steerable drilling device including steering
19%
– Miscellaneous 12%

It should be noted here that it was not the objective of the initial trial to drill as fast as possible, but to prove the validity of the chosen concept.

After removal of the steerable drilling device, a wire-line caliper and deviation survey, at 6" (152 mm) intervals was carried out. No significant deviations were detected, as all deviations were under the tolerance range of the measuring equipment.

6.2 SECOND TRIAL

After this positive initial trial a further application was carried out within the Makejewka ventilation shaft drilling project in the Donezk coal area (fig. 29).

This drilling had commenced already with a conventional “bottom hole assembly” (cutter head, stabilizers and drill collars) with a drilling diameter of 4.0 m (13,13'), and had reached a depth of 300 m (984,25'). At this level the deviation was 470 mm (18,51"). To continue this shaft with the steerable drilling device, it was necessary to first drill a central starting hole of 2.10 m (6,89') dia. below the portion of hole of 4.0 m (13,13') dia. already drilled. The proposed bottom hole assembly configuration is shown in fig. 30. Due to the fact that the drill hole was not straight in the upper 300 m (984,25'), the starting hole could only be drilled with one rather than two stabilizers. A starting hole of approx. 3.20 m (10, 5') length was considered sufficient to accomodate the lower clamping system of the steerable drilling device.

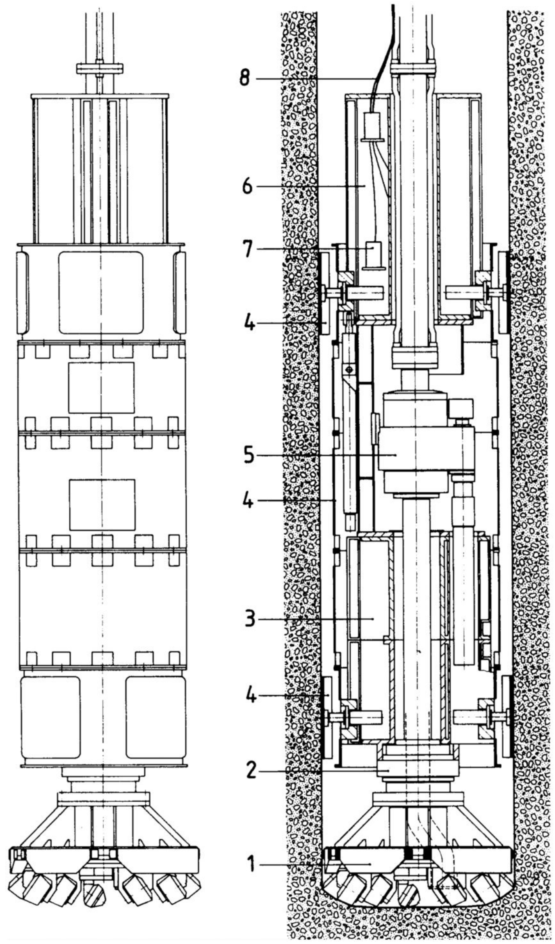


fig. 25: the main components of the steerable drilling device (SDD) (scheme)

The cutter head of 2.1 m (6,89') dia with only one stabilizer wandered to all sides so that the starting hole dia. was larger than intended.

After finalizing the preliminary work, the assembly of the steerable drilling device was carried out. The steerable drilling device could not be clamped correctly in the oversize drill hole with the lower clamps, and the outer kelly slipped down. Thus, initially steering was not possible and the deviation increased over the first few meters to approx. 505 mm (19,89").

As a result, collaring had to be effected very cautiously as at 100–200 kN (22456, 68 lbs− 44913,36 lbs) thrust the penetration rate amounte to only 6–15 cm/h (2,37–5,91"/h).

The upper clamping and steering level could be actuated for steering after drilling a total of 5.9 m (19,36') below the start at 300 m (984,25' From that point, drilling was carried out at approx. 20–30 tons (22–33 short tons) thrust The max. penetration rate amounted to approx. 25 cm/h (9,25"/h).

The steerable drilling device was used over a depth of about 30 m (98,43'), under geological conditions which are characterized as follows:

- soft to medium-hard rock formations
- strata thicknesses between 0.2 to 20 m (7,88" to 65, 65'), steadily alternating
- high clay content
- strata inclination from horizontal to 15 degrees

This demonstrated that the desired steerability of the device was achievable. The direction of deviation predetermined in the upper range of the shaft was cautiously corrected and from depth 306 m to depth 332 m (1004' to 1089') there was a deviation increase of less than 25 mm (1").

The hydraulic system, the pressure compensating system, the packings, and the measuring and steering devices worked satisfactorily during the test.

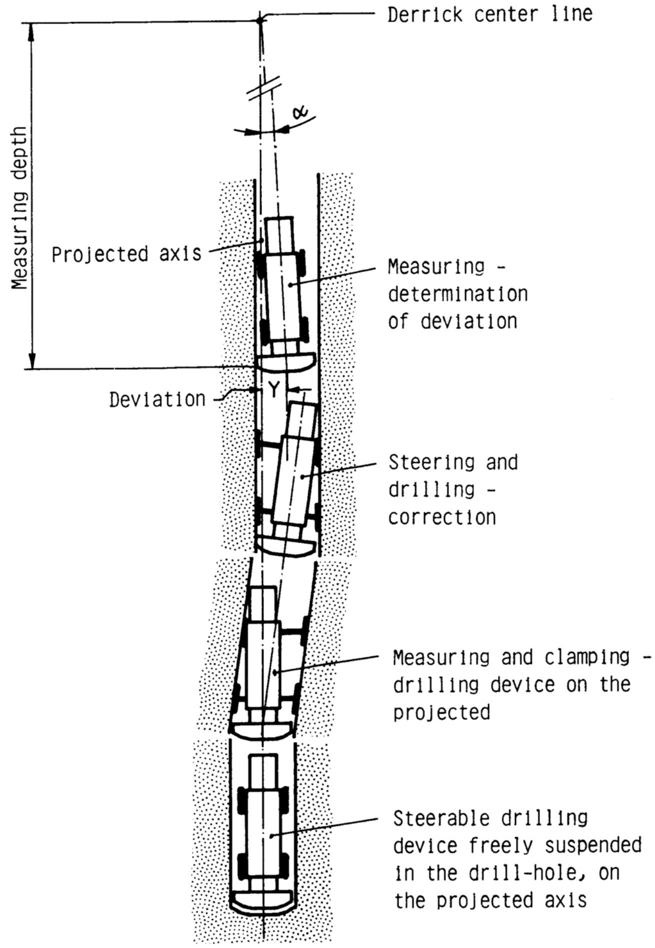


fig. 26: measuring and steering operations with the SDD

Subsequently, the drill hole of 2.1 m (6,89') diameter was reamed by a reaming head to 4.0 m (13,13') dia. by the air-lift procedure using a guiding device especially developed for that purpose (fig. 31, 32).

At a load of approx. 35 tons (38,47 short tons), drilling rates of approx. 0.4 m/h (15,75"/h) were achieved.

This reaming was effected without any incidents and has demonstrated on a small scale that reaming of a straight course pilot hole can be achieved without any problem.

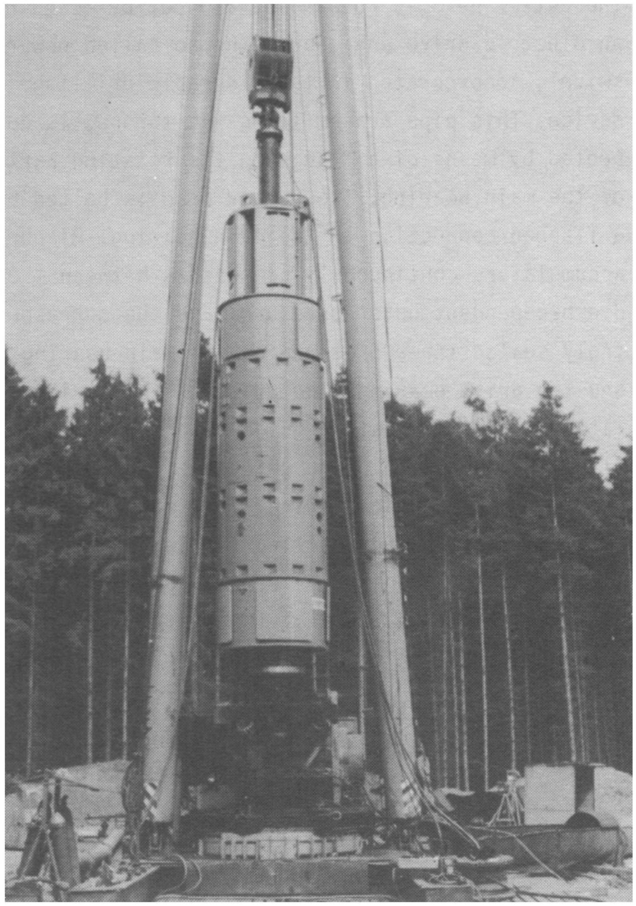


fig. 27: SDD at “Hambacher Forst”

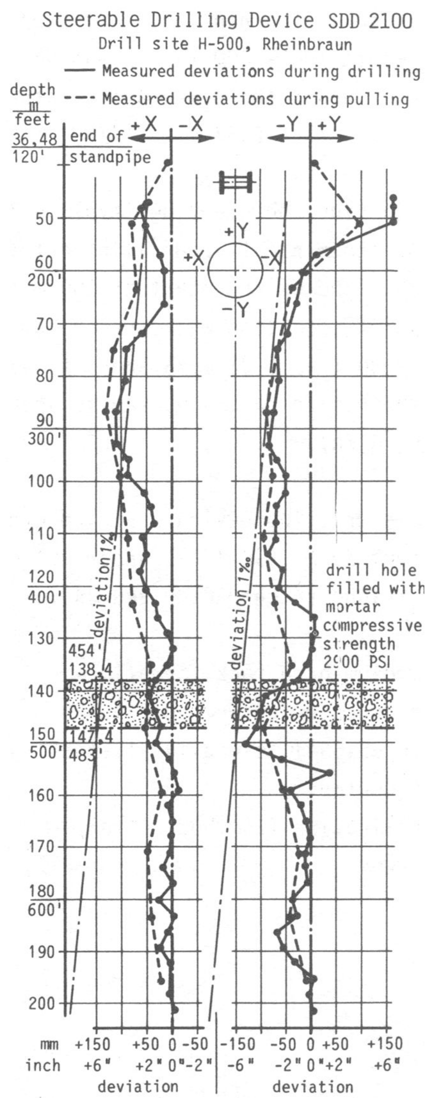


fig. 28: SDD-deviation diagramm “Hambacher Forst”

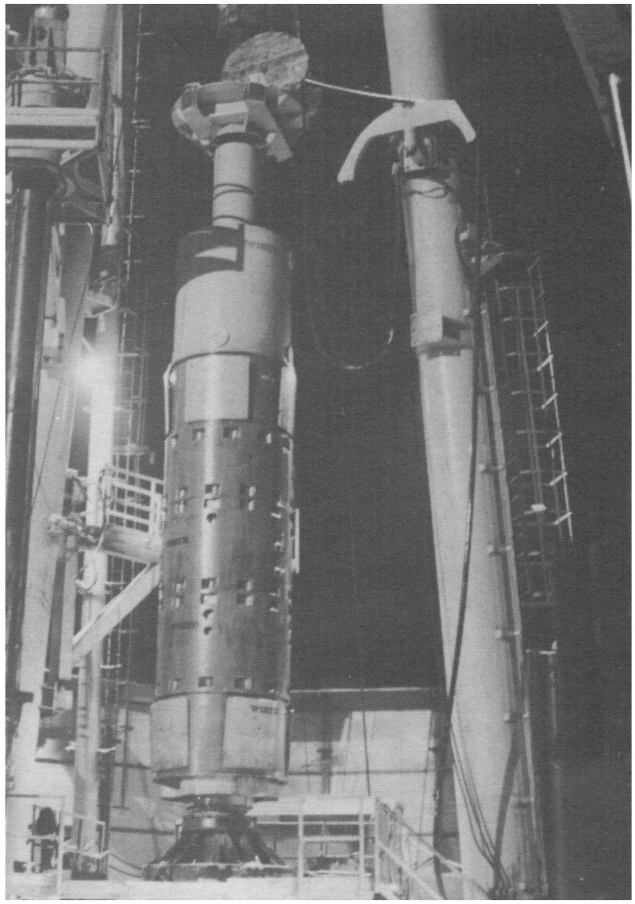


fig. 29: SDD at Donezk

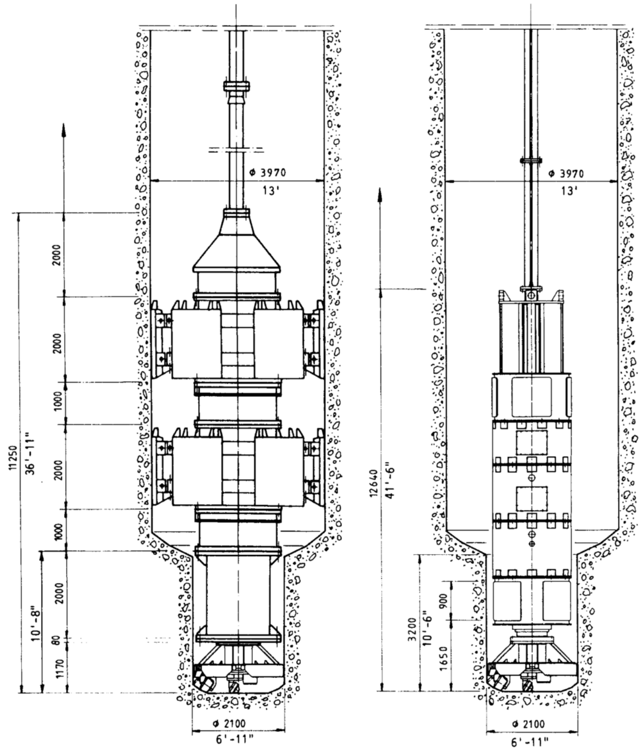


fig. 30: drilling the starting hole for and starting drilling operation of the SDD

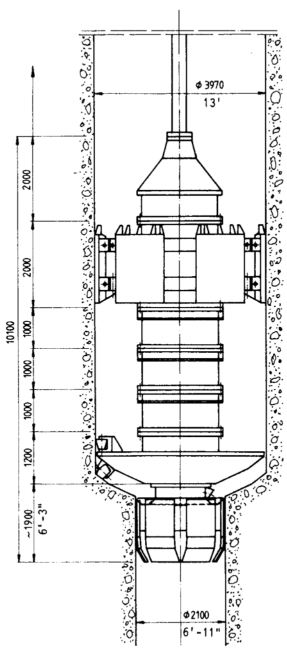


fig. 31: reaming operation of the pilot hole drilled by the SDD

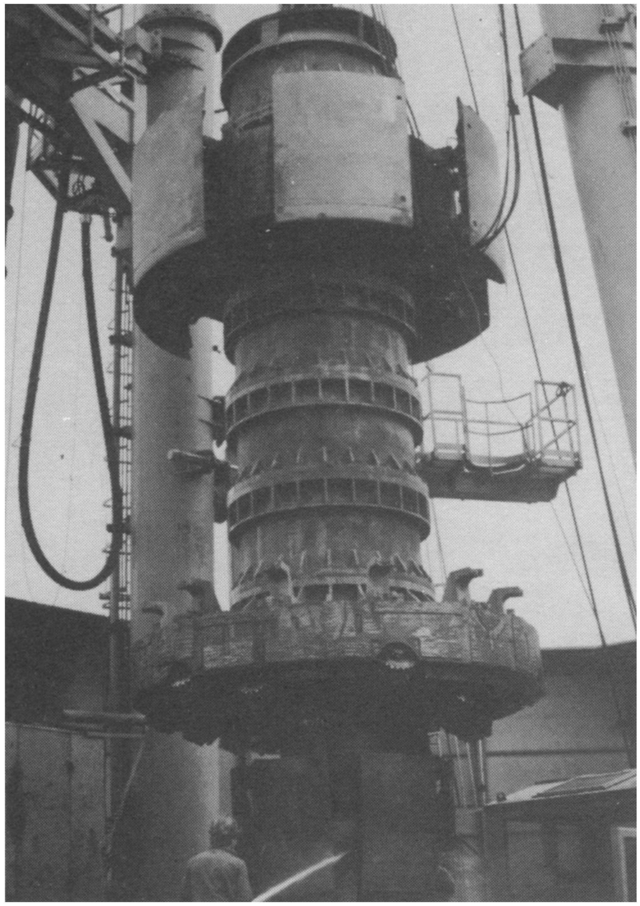


fig. 32: the bottom-hole-assembly for the reaming operation

High-strength, superior durability, concrete shaft linings

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SYNOPSIS

All of the deep shafts sunk from the surface in the U.K. since 1977 have been for British Coal and they have all been concrete lined[1]. The first part of the paper is a review of the aspects involved in producing good quality, strong and durable concrete shaft linings during the construction of the Selby New Mine Complex over the period 1977 to 1986. Shaft lining details are given including the strata sunk through, concrete lining specifications and mix designs. Construction factors refer to workability requirements, concrete production, transportation and placing. Special conditions are discussed which cover the casting of concrete against frozen ground and the use of cement replacement materials to reduce the possibility of thermal cracking in thick shaft linings. The first part of the paper is concluded by emphasizing the improvements in concrete technology which have been achieved during the installation of the linings at Selby.

Most of the shaft lining concrete at Selby had a maximum specific design strength of 45 N/mm^2. However to resist hydrostatic pressures down to 690 m at Riccall and 660m at North Selby, characteristic strengths of 60 N/mm^2 were required. This was achieved using sulphate resisting Portland cement (SRPC) concrete and cement replacement (Cemsave slag) concrete, but with lining thicknesses of 1400 mm in shafts of 7.315 m internal diameter. Subsequent to Selby, further advances in shaft lining concrete technology have been made.

Higher strength linings of 70 to 80 N/mm^2 characteristic strength have now been produced which, in addition to permitting the use of thinner linings to greater depths, exhibit very much improved long term durability. In the second part of the paper, details are provided on British Coal's Maltby No.3 shaft lining which was constructed during the period 1981–87. The problem of possible chloride attack from the strata water is outlined and the first U.K. use of microsilica concrete underground to combat this problem is discussed in relation to its incorporation in the sump section of the Maltby No.3 shaft lining during November 1987.

Benefit from the high strength and superior durability of microsilica concrete has also been obtained by using the material in the lining of British Coal's Asfordby Mine upcast shaft. This shaft was installed during the period of 1985–88 and the third part of the paper concentrates on the use of microsilica concrete in the sump section of the shaft lining at Asfordby in 1988. Finally, the paper is concluded by commenting on which direction should be followed in relation to the production of future high strength, superior durability, concrete shaft linings.

THE SELBY MINE COMPLEX

The total project consists of ten shafts located in pairs at the Wistow, Stillingfleet, Riccall, Whitemoor and North Selby satellite mine sites, They provide men and materials access, together with ventilation, for the extraction of ten million tons of coal per annum from the Barnsley Seam, under the Vale of York, via twin parallel drifts reaching the surface at Gascoigne Wood. Cementation Mining Limited were awarded contracts for the shafts at Wistow, Riccall and North Selby. Shaft construction commenced in 1977 at Wistow and was completed in 1986 at North Selby.

Shaft lining details

The shafts are all 7.315 m in finished diameter and concrete lined to depths of 415 m, 708 m, 802 m, 965 m and 1033 m at Wistow, Stillingfleet, Riccall, Whitemoor and North Selby respectively. Ground freezing was employed at all sites from the surface to prevent ingress of water from the Bunter Sandstone during sinking, the corresponding freeze depths being 148 m, 165 m, 253 m, 305 m and 283 m respectively. The freeze depth was extended to 273 m in Wistow No.1 shaft to reach the unstable Basal Sands. Figure 1 gives details of the concrete shaft linings installed in the shafts at Wistow, Riccall and North Selby. The illustration shows how the concrete linings vary in thickness with depth to resist the imposed hydrostatic pressure; it shows the strata sunk through and also indicates the lining specified concrete strength grades and required sulphate resistance categories. Shaft lining concrete mix designs for Wistow, Riccall and North Selby are detailed in Table 1. A total of approximately 170,000 m^3 of concrete were placed in the ten shaft linings at Selby, most of it high strength, high quality concrete placed in a restricted environment very successfully with the aid of plasticising and superplasticising admixtures.

Shaft lining construction

Figure 2 shows the scaffold suspended in the shaft from which all sinking and lining operations were carried out. It also indicates the lining pouring sequence. The method of transporting concrete from surface to the scaffold was the standard shaft sinking practice of free-fall under gravity down a 150 mm or 200 mm diameter steel pipe. A dash pot, located at the bottom of the pipe, acted as a receiving and remixing vessel from which the concrete flowed to a distribution box on the top deck of the scaffold and thence via flexible hoses directly into the shaft lining forms. Due to the types of concrete mix now being employed which are highly workable, cohesive and possess non-bleed characteristics obtained with the help of plasticising and superplasticising admixtures, this method is not detrimental whatsoever to the concrete and high quality concrete was placed to a depth of over 1000 m at North Selby successfully by this method.

The required concrete mix workability was in the range of 60–75 mm without the inclusion of a plasticiser. On adding the plasticiser, the expected slump when measured at the shaft top was in the order of 160 mm and greater. The aim was to achieve a minimum slump of 150 mm at the point of placing, taking into account a slump loss of approximately 25 mm per 300 m of shaft depth. Hence concrete with a slump approaching 200 mm was needed for the lower sections of the 1000 m deep shaft linings.

At each of their shaft contract sites, Wistow, Riccall and North Selby, Cementation Mining Limited sub-contracted the supply of concrete to ready mixed concrete suppliers who established their own batching plants on site adjacent to the shaft tops. Transport over the short distance from batching plant to shaft top was by rotating drum lorries. The quality control of an established ready mixed concrete batching plant therefore ensured that a high quality concrete was provided for shaft lining construction.

Special conditions

To be able to construct good quality, strong and durable shaft linings in the underground environment, many difficulties have to be overcome. Not only is it harder to transport and place concrete in the confined situation of a shaft, when compared with the open aspect of above ground construction, other adverse conditions prevail. At Selby, it was necessary to place concrete directly against frozen ground in the upper section of the shafts. This, however, did not present much of a problem because the linings were of sufficient thickness (minimum of 600 mm) that, when cast, enough heat of hydration was developed to overcome the frost action until the concrete had matured.

Another aspect of paramount importance was being able to install a concrete lining which was resistant to attack from sulphates in the strata water. Figure 1 shows the required shaft lining sulphate resistance categories at

Table 1: Shaft lining concrete mixes (selby)

Site	Selby Wistow	Selby Riccall	North Selby	North Selby
Specified concrete	Grade 45 with sulphate resisting Portland cement (SRPC)	Grade 45 SRPC to Class 4 B.R.E. Digest 174	Grade 45 SRPC to Class 4 B.R.E. Digest 174	Grade 45* cement replacement (Cemsave slag)
Supplier	Topmix Ltd.	Trumix Ltd.	Topmix Ltd.	Topmix Ltd.
Cement (or total cementitious content) (kg/m^3)	Ribblesdale 420	Rugby Crown 460	Blue Circle 440	500 (30% OPC, 70% Cemsave)
Sand (kg/m^3)	Elvaston Zone 2	Farnham Zone 2	Blaxton Zone 3	Blaxton Zone 3
	615	665	670	595
Sand % of total aggregate	35	39	37	34
Coarse aggregate (kg/m^3)	Elvaston gravel 1140	Farnham gravel 10mm 350 20mm 700	Blaxton gravel 1120	Blaxton gravel 1150
Water (1/m^3)	180	186	185	180
Water: cement (or total cementitious content) ratio	0.43	0.40	0.42	0.36
Slump without plasticiser (mm)	75	75	75	60

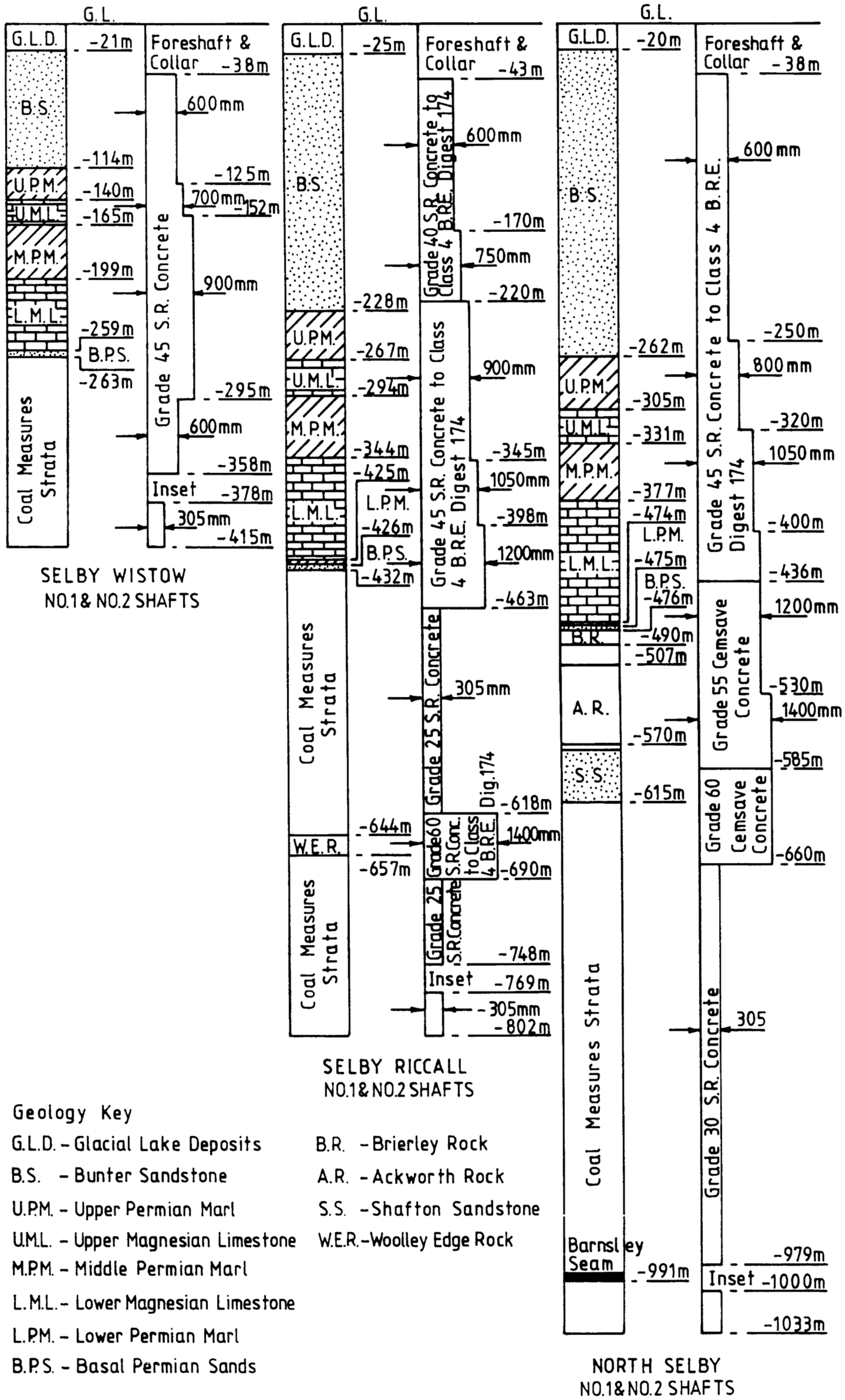

Figure 1 Geology, shaft lining thicknesses, specified concrete grades and required sulphate resistance categories for the Selby shafts

Site	Selby Wistow	Selby Riccall	North Selby	North Selby
Plasticiser	Flocrete N, 0.18 to 0.36 litres per 50 kg cement			
Slump with plasticiser	Varies with shaft depth but generally 160mm and upwards			

* Upgraded to Grade 55 based on test cube strengths achieved of about 60 N/mm^2

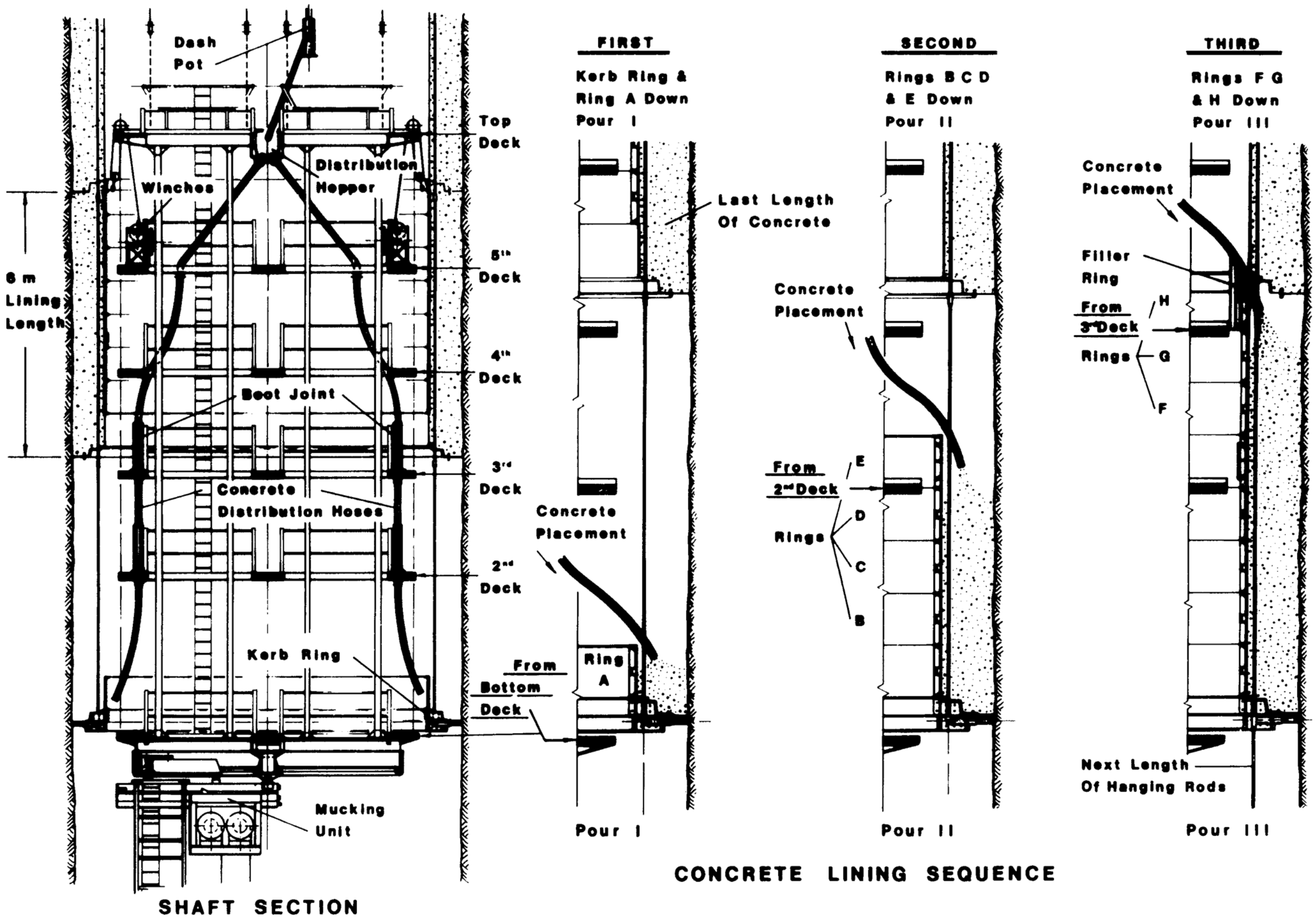

Figure 2 Shaft lining construction

Selby in relation to the concrete grades. The problem was exacerbated at depth, where thick shaft linings were required to resist the high water pressures, as it was considered that the thicker walls could be more prone to thermal cracking. During the early months of 1981, discussions took place between British Coal (the National Coal Board at the time), W.S. Atkins and Partners, Frodingham Cement Co. Ltd. and Cementation Mining Ltd. with a view to introducing Portland blast-furnace cement (PBFC), based on Cemsave ground granulated blast-furnace slag and ordinary Portland cement (OPC), into the concrete mix design for the shaft linings at North Selby in place of sulphate resisting Portland cement (SRPC). The reasons for the introduction were:

1) its lower heat of hydration temperature rise and hence alleged lower susceptibility to thermal stressing and cracking;
2) its greater sulphate resisting capacity, in this case in the form of 30%:70%-OPC: Cemsave to give the required Class 4 sulphate resistance[2];
3) its supposed economy in terms of lower cost per cubic metre of concrete produced.

After several months of discussion and testing, the use of Cemsave was accepted on a mutually agreed basis between British Coal and Cementation Mining Ltd.

On completion of a number of pours incorporating Cemsave it was noticed that in some pours vertical hairline cracks had appeared. Further investigation into stripping times seemed to indicate a possible link to length of time the formwork remained in position on the wall before being released, the longer the time period the more predominant were the cracks in that pour. No quantitative study had been made to confirm or refute such supposition and a calculation exercise was carried out in an attempt to define more closely actual thermal stress levels in a shaft wall restrained by the external strata and the internal formwork. The results of this exercise are contained in Figure 3.

Both the SRPC mix and the PBFC mix are specified in Table 1. The upper set of curves in Figure 3 indicate the variation in cube strength with age for both mixes. These results were obtained from cubes subjected to temperature matched curing, the temperatures being recorded in a concrete test block with simulated rock and steel formwork to represent the shaft situation.

In the second set of curves an assumed relationship between cube strength and modulus of elasticity, E, was taken to relate the two properties[3]:

$$E = 5.9 \sqrt{\text{cube strength}}$$

It was necessary to know this variation of E to be able to convert thermal strains into stresses. Temperatures developed within the shaft wall and related to time are shown in the third set of curves in Figure 3. The PBFC results are actual results from thermocouples installed in the North Selby shaft lining whereas the SRPC results were adjusted from the matched curing temperatures recorded in the test block. The in situ shaft results for the PBFC mix were slightly higher than the equivalent test block values and hence the adjustment for the SRPC mix. Using a coefficient of expansion of 12×10^{-6} per °C for concrete with gravel aggregate[4], from the temperature profiles thermal strains were computed and converted to stresses on an incremental basis using average values of E over each increment. These results are plotted in the fourth series of graphs. It can be seen that during the period of increasing temperature, compressive stresses are developed due to the "locking in" action of the surrounding ground which prevents expansion. On cooling, subsequent to releasing the "locked in" compressive stresses, tensile stresses could develop if the formwork acted as a restraint.

However, reviewing the strains and stresses developed, normal concrete cracks at strains in the order of 100–200 microstrain[5]. At four days, for a concrete strength of approaching 40 N/mm^2 and an E value of about 35 kN/mm^2, the failure tensile strength range would be 3.5–7 N/mm^2. Looking at the bottom set of curves in Figure 3, it could be expected that both types of concrete would be prone to thermal cracking if the formwork created an inner restraint. Why then, from experience, is the SRPC mix not prone to vertical cracking whereas the PBFC mix indicated such a trend?

One postulation can be proposed. For a steel formwork coefficient of thermal expansion equal to 13.3×10^{-6} per °C[6], which is not so far different from that of the concrete, it could be expected that thermal strains in both the concrete and steel would be similar. Radially, heat conduction through the thin steel form would be rapid and therefore the temperature of the form would remain close to that of the inside face of the concrete. Being thermally strain compatible with the inside face of the concrete, the form should not provide a restraint. The thermal contraction takes place by way of an unrestrained radial movement of all points inwards, without inducing tensile strain and stresses in the circumferential direction. From a closer inspection of the third set of curves in Figure 3, it can be seen that, at a four to five day age, the SRPC centre and face temperatures, together with the PBFC face temperature, are all close to ambient. However, due to the slower dissipation of heat from the centre of the wall in the case of the PBFC mix, a large fall in temperature still has to occur in the centre over subsequent days. The formwork, already at ambient temperature, will then act as an inner restraint, preventing inward radial thermal contraction and converting it into circumferential tensile strain and stress with a resulting potential for vertical cracking in the concrete.

On the basis of the above hypothesis, where cement replacement concrete mixes are used in shaft linings, which dissipate the heat of hydration from the centre of the wall over a longer period than for normal mixes, it is important to release the formwork at an early age to avoid the possibility of vertical cracking in the concrete due to thermal contraction. In relation to North Selby, the few cracks which appeared were hairline, none were considered detrimental to the overall structural integrity of the lining and all were sealed successfully by cement backwall grouting.

Improvements in concrete technology

In the twenty years prior to the advent of Selby in 1977, only a few large diameter, deep shafts had been sunk in the U.K. Daw Mill Colliery No.2 shaft was the last for British Coal, which was constructed by Cementation Mining Limited during 1968–70. Additional to the long previous period of inactivity, the Selby shaft lining high concrete strengths specified were new to normal shaft sinking practice. The successful installation of the large quantities of high quality, high strength concrete in the confined underground shaft environment was largely due to three aspects:

1) The skill of the site engineers and operatives.
2) The use of plasticisers, retarding plasticisers and super plasticisers to improve the concrete workability, which at the same time permitted the water/cement ratio to be kept relatively low to guarantee the strength and produced cohesive, non-bleed mixes.
3) The establishment of ready mixed concrete suppliers' batching plants close to the shafts. Knowledge of the performance of their own materials, together with accredited quality control procedures, guaranteed the delivery of a high quality product to the shaft top.

With regard to the latter, because of the quality achieved at North Selby, it was possible to upgrade the Grade 45 SRPC mix to a Grade 55 based on test cube strengths approaching 60 N/mm^2.

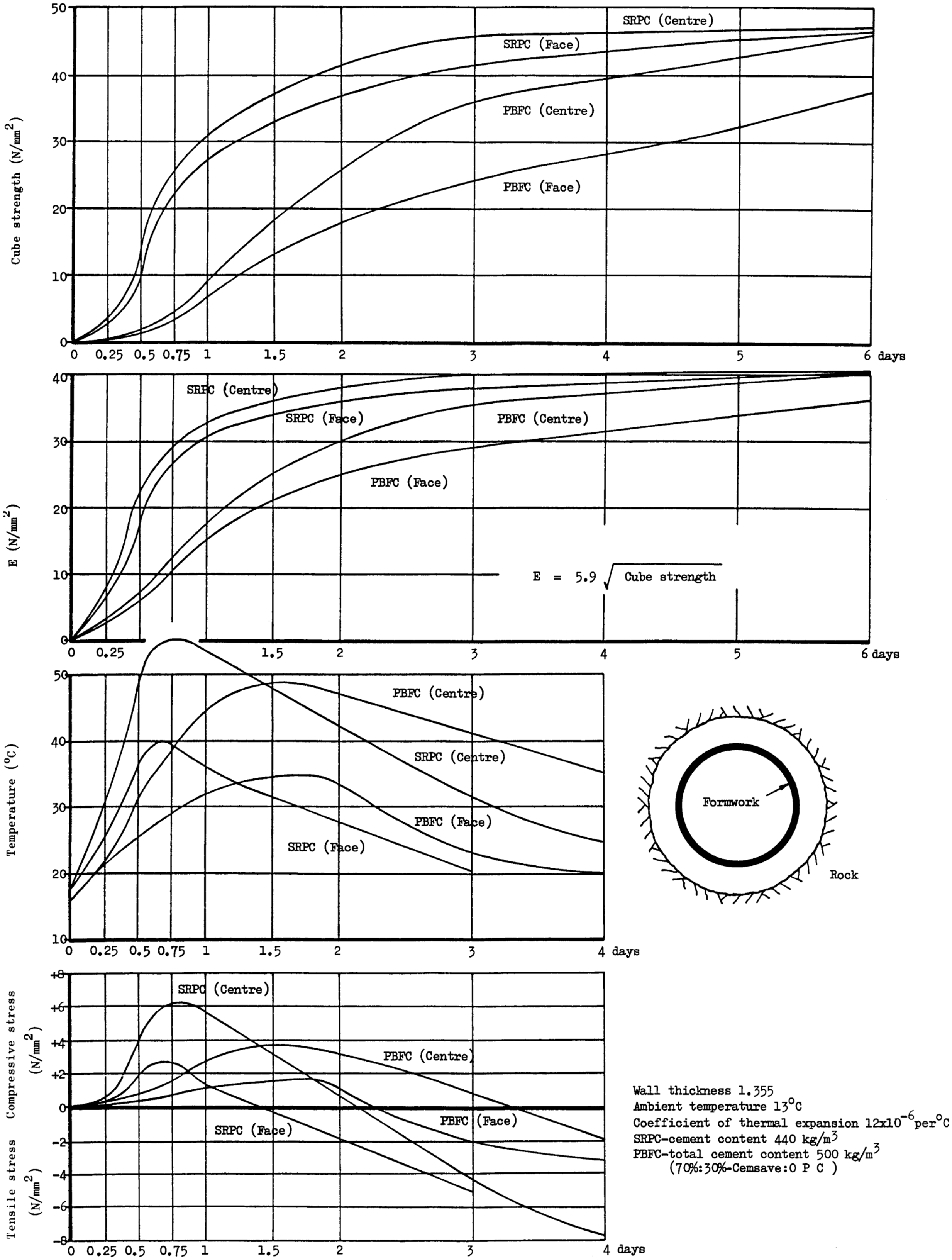

Figure 3 Thermal stresses in concrete shaft linings with restraint from external strata and internal formwork

Credit should also be given to British Coal for being prepared to consider and permit the use of alternative concrete mixes, such as the North Selby cement replacement (Cemsave slag) mix, with a view to providing improved strength and durability properties in shaft linings.

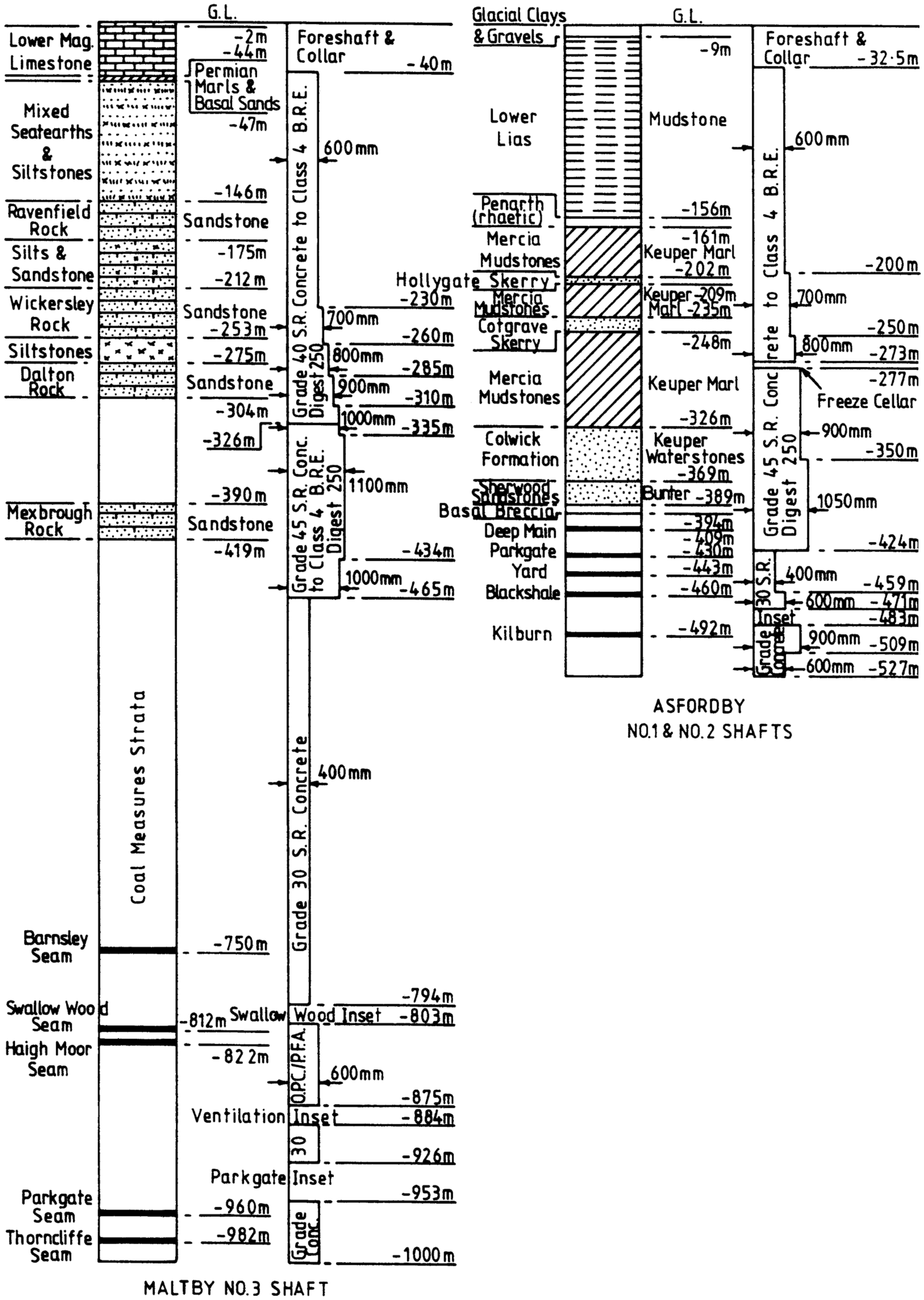

Figure 4 Geology, shaft lining thicknesses, specified concrete grades and required sulphate resistance categories for the Maltby and Asfordby shafts

Table 2: Shaft lining concrete mixes (Maltby and Asfordby)

Site	Maltby	Maltby	Asfordby
Specified concrete	Grade 45 SRPC to Class 4 B.R.E. Digest 250	Grade 30 cement replacement (OPC/PFA)	Grade 45 SRPC to Class 4 B.R.E. Digest 250
Supplier	Ready Mixed Concrete Ltd.	Ready Mixed Concrete Ltd.	Redland Readymix Ltd.

Site	Maltby	Maltby	Asfordby
Cement (or total cementitious content) (kg/m^3)	Ketton 460	430 (300 OPC :130 PFA)*	Ketton 420
Sand (kg/m^3)	Butterley (Thurlby) Zone 2/3 557	Butterley (Thurlby) Zone 2/3 727	Besthorpe Grade M 770
Sand % of total aggregate	33	40	43
Coarse aggregate (kg/m^3)	Redland (Besthorpe) gravel 20–5mm 1156	Redland (Besthorpe) gravel 20–5mm 1079	Mountsorrel granite 10mm 400 20mm 620
Water (1/m^3)	184	170	181
Water:cement (or total cementitious content) ratio	0.40	0.40	0.43
Slump without plasticiser (mm)	50	50	75
Plasticiser	Flocrete N, 0.18 to 0.36 litres per 50 kg cement		
Slump with plasticiser	Varies with shaft depth but generally 160mm and upwards		

* Ketton OPC and Pozzolanic (Eggborough) PFA

MALTBY

Shaft lining details and construction

Details of British Coal's Maltby Colliery No.3 shaft are given in Figure 4. This shaft has an 8m internal diameter and was constructed by Cementation Mining Limited during the period 1981–87. No ground freezing was needed to prevent the inflow of strata water into the excavation during sinking as the estimated potential inflow was low. Only a small amount of forward strata grouting from the sump, during sinking, was required in some areas to reduce the water to a minimum for handling. Nevertheless, the lining was designed to resist hydrostatic pressure to a depth of 465 m to cater for any build up of water pressure once the lining had been installed and backwall grouted up. The construction method was similar to that detailed for Selby. Concrete mix designs for Maltby are given in Table 2.

Resistance to chloride attack

One of the big problems with the Maltby No.3 shaft, in relation to producing a durable concrete lining, was the presence of high quantities of dissolved chlorides solids in the strata water below the Barnsley Seam. Typical figures from an analysis of the strata water were in the order of 110000 to 125000 mg/1 of dissolved chlorides solids. This produced a very aggressive environment in which to install concrete linings. Sulphate resistance was the predominant requirement for

Table 3: Microsilica concrete mixes (Maltby and Asfordby)

Site	Maltby (Trial)	Maltby (Production)	Asfordby (Trial)
Supplier	Ready Mixed Concrete Ltd.	Ready Mixed Concrete Ltd.	Redland Readymix Ltd.
Total cementitious content (kg/m^3)	335 (200 Densit Binder*:135 Ketton OPC)	350 (315 Ketton OPC:35 microsilica**)	350 (315 Ketton OPC:35 microsilica**)
Sand (kg/m^3)	Butterley (Thurlby) Zone 2 810	Butterley (Thurlby) Zone 2 805	Redland (Besthorpe) Grade M 870
Sand % of total aggregate	42	42	47
Coarse aggregate (kg/m^3)	Redland (Besthorpe) gravel 20–5mm 1120	Redland (Besthorpe) gravel 20–5mm 1110	Redland (Mountsorrel) granite 10mm 370 20mm 610
Water (1/m^3)	140	162	144
Water: cementitious content ratio	0.42	0.46	0.41
Slump (mm)	210	–	190
Flowtable spread (mm)	485	600	510

Site	Maltby (Trial)	Maltby (Production)	Asfordby (Trial)
Plasticiser	Powder in Densit Binder+ 3 litres/m^3 of Supaf lo R	6:1 NS***+ Flowcrete Rl 7 litres/m^3	6:1 NS***+ Flocrete Rl 7 litres/m^3
24 hr strength (N/mm^2)	24.3 (mean of 3 cube results)	–	–
28 day strength (N/mm^2)	83.3 (mean of 9 cube results)	74 (mean of 22 cube results)	86 (mean of 2 cube results)

* Densit Binder=5:1 cement: microsilica+powdered super plasticiser giving microsilica content in mix of 10%
** Carbon Enterprises Ltd. densified microsilica powder (10% content)
*** Naphthalene sulphonate

durability above the Barnsley Seam and sulphate resisting Portland cement (SRPC) concrete was specified accordingly (see Figure 4). Below the Barnsley Seam, sulphates in the strata water were almost non-existent and the concrete specification was changed to an ordinary Portland cement (OPC)/pulverized fuel ash (PFA) mix to provide more resistance to the aggressive chlorides.

Microsilica concrete

In December 1985, Cementation Mining Limited signed an exclusive Licence Agreement with Densit A/S, a subsidiary company of Aalborg Portland in Denmark, to market and use Densit materials for mining applications in the U.K. Densit is the trade name of a group of new cement based materials exhibiting ultra high strength, extremely low permeability and superior durability. Densit concrete is made from ordinary or high strength concrete aggregates, a special cement named Densit Binder and water. Densit Binder is a premixed blend of cement, microsilica and chemical admixtures in powdered form. It can be supplied in bulk or in bagged form and is British Coal approved for underground use (APP NO A 3203). With this background, plus a request from British Coal to look at the possibility of using cement replacement concrete mixes for the lower section of shaft lining in order to provide greater resistance to chloride attack, Cementation Mining Limited suggested that an OPC/microsilica mix should be considered in addition to an OPC/PFA mix. Using the site ready mixed concrete batching plant, a first set of trial mixes was carried out on three mixes:

1) Grade 30 OPC/PFA (see Table 2)
2) OPC/Densit Binder (see Table 3)
3) OPC/microsilica −90%:10%

The first mix presented no problems to the standard production procedure, using the available cement silos, and performed satisfactorily. With the other two mixes, the Densit Binder and microsilica were discharged into the mixer in the trials from bags as supplied. The OPC/Densit Binder mix gave excellent workability and strength results (see Table 3). However, for the OPC/microsilica mix, although reasonable strengths were achieved, satisfactory workability could not be obtained when the densif ied microsilica on its own was added directly to the mix.

Resulting from the relatively high cost of producing a Densit concrete, compared with the cost of introducing microsilica on its own directly into the mix, British Coal opted for the adoption of the OPC/PFA mix in the lower section of the shaft, with an increased wall thickness as an additional durability safeguard (see Figure 4). Cementation Mining Limited had, in the meantime, carried out a further set of trial mixes through the batching plant which proved that, with the correct superplasticisers, microsilica could be added directly to the mix and dispersed satisfactorily to give both the required workability and high strength. As a result of this, British Coal agreed to permit the installation of a trial length of lining in the shaft sump. Approximately 720 m^3 of the production mix microsilica concrete (see Table 3) were placed in the sump floor slab and walls up to a height of 12 m from the top surface of the floor slab. The concrete was produced with very little difficulty through the normal ready mixed concrete batching plant system with bulk tanker delivery of the densified microsilica into one of the cement silos.

Although very high strength was not a requisite for the Maltby shaft lining, the extremely low permeability of the microsilica concrete was regarded to be a benefit as it would reduce the susceptibility of the concrete to chloride ion diffusion and hence provide improved durability. Figure 5 contains the results of permeability tests carried out on cubes of the various trial mixes. It can be seen that the Densit concrete gives the greatest reduction in permeability, by almost 10^2 in relation to the standard SRPC shaft mix.

ASFORDBY

Shaft lining details and construction

Details of British Coal's two Asfordby mine shafts are contained in Figure 4. Both shafts, 7.32 m in finished diameter, were constructed by Cementation Mining Limited during 1985–88. The ground was frozen between 277 m–405 m to prevent ingress of water from the Colwick Formation and Sherwood Sandstones. This was achieved by drilling the freeze holes and installing the casings from the surface and interconnecting them up with a brine main installed in a freeze cellar at a depth of 277 m. The brine main was fed from surface by vertical inlet and return pipes fixed in the shaft. Design of the shaft linings was carried out on a hydrostatic pressure resisting basis to a depth of 424 m. The construction method was the same as for the shafts previously described and the concrete lining mix design is given in Table 2.

Strength and durability requirements

Discovery of an igneous intrusion closer to the upcast shaft than expected in the sump area, with a high water inflow potential, meant raising the sump bottom position above its original level of 527 m. As an added safeguard, the shaft lining in the sump area was designed to resist the hydrostatic pressure at that depth, which required a Grade 75 concrete with a wall thickness of 1000 mm. Cementation Mining Limited's experience with high strength, microsilica concrete mixes enabled this strength and wall thickness to be considered as a practicable solution and trial mixes were successfully carried out on site using the ready mixed concrete batching plant (see Table 3). Currently 665 m^3 of production concrete have been placed in the shaft sump section of lining over a length of 16.1 m at a depth of 447.2 to 463.3 m below collar level. Another 900m^3 will eventually be placed completing a total length of 40.1m. The concrete was batched through the normal operating procedure of the ready mixed concrete plant, with bulk tanker delivery of the densified microsilica into one of the cement silos. From the cube test results obtained to date, the mean strength achieved was 81 N/mm^2 and this, together with the inherent very low permeability of the microsilica concrete, characterizes the trend towards high strength, superior durability, concrete shaft linings.

THE WAY AHEAD

It was mentioned in the Synopsis that the current practicable limit on depth for hydrostatic pressure resisting concrete linings was about 650 m for a 7.315 m diameter shaft. This required a lining thickness of 1400 mm with a Grade 60 concrete, the latter approaching the upper strength limit for normal concrete. To be able to resist water pressures at greater depths and correspondingly install slimmer shaft linings, without resorting to other more costly materials and construction methods such as welded steel plates, higher strength concrete is a must. It has been proved that microsilica concrete offers an economical solution to the problem. The high strength, together with its superior durability, should make this material popular for future shaft linings.

REFERENCES

1. Auld F.A. A decade of deep shaft concrete lining. Concrete, vol. 21, No.2, February 1987, p. 4–8.
2. Building Research Establishment. Digest 250 (superseded 174). Concrete in sulphate bearing and groundwaters. Her Majesty's Stationery Office, London, June 1981.
3. Comité Européen du Beton-Federation Internationale de la Précontrainte (CEB—FIP). International recommendations for the design and construction of concrete structures. Principles and Recommendations. June 1970: FIP Sixth Congress, Prague. p. 27.
4. British Standards Institution. BS 8110: Part 2:1985. Structural use of concrete. Part 2. Code of practice for special circumstances. Section 7.5. p. 7/4.
5. Bache H.H. Introduction to compact reinforced composite. Nordic Concrete Research Publication No.6, Paper 2. Nordic Concrete Feceration, Oslo. 1987. p. 3.
6. Rollason E.C. Metallurgy for engineers, Edward Arnold, London, 1959. p.298.

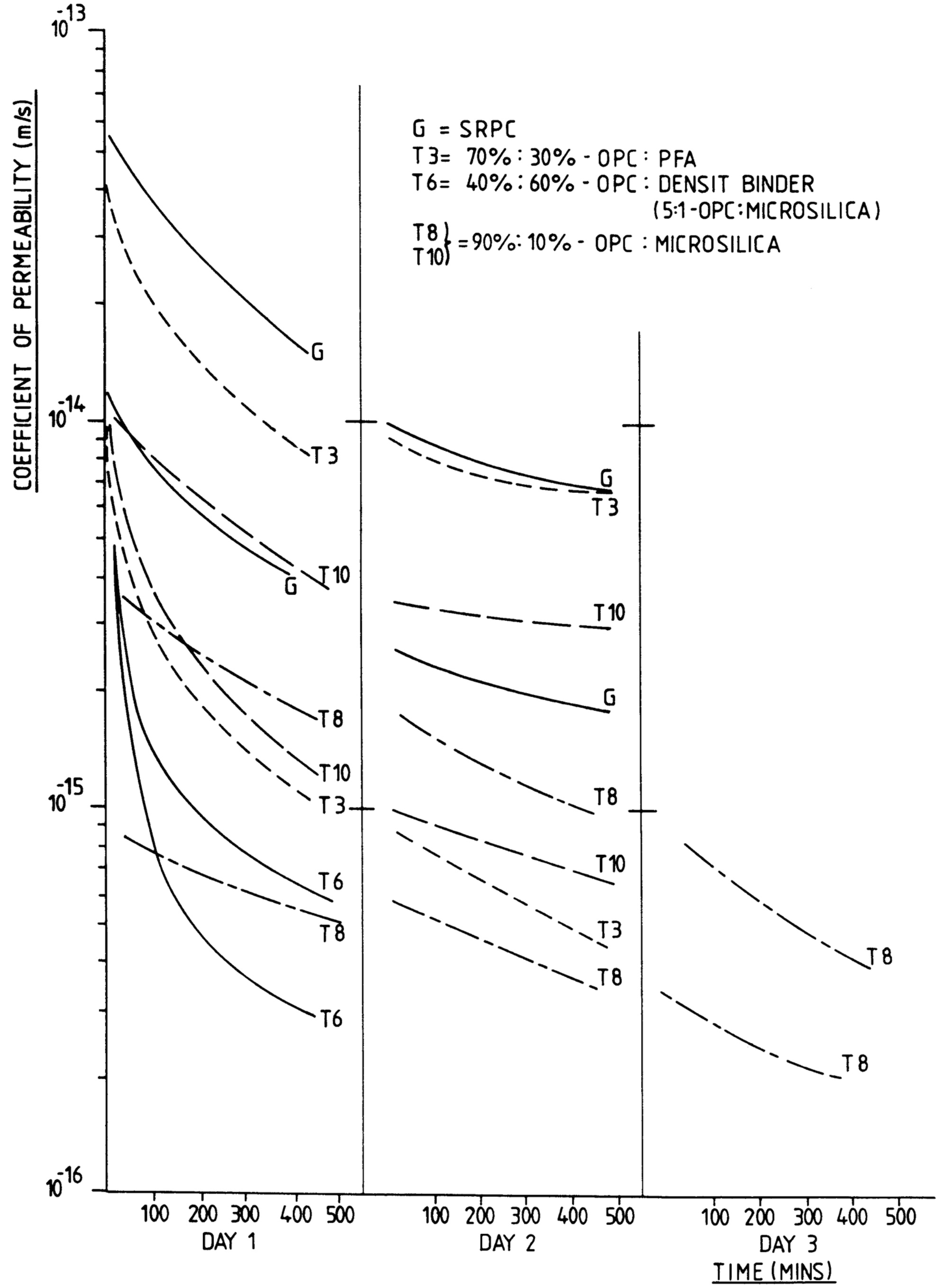

Figure 5 Test concrete permeability reduction due to microsilica incorporation—Maltby trials (by courtesy of TH Technology R.& D. Centre)

Deep repository shaft design—offshore and onshore concepts

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SYNOPSIS

The objective of UK Nirex Ltd is to develop a single deep mined repository for the safe disposal of low and intermediate level radioactive waste. One component of the Company's work over the past 12 months has been to assess the costs and constraints associated with repository construction and operation in a range of host rocks and settings.

This paper describes the concepts considered, with particular emphasis being given to the design of the shafts or drifts which would provide access to the underground works and presents the conclusions of design work undertaken by specialist contractors commissioned by Nirex. The conclusion from these studies is that a deep repository could be constructed and operated safely in any of the favoured geological environments, although each offers particular advantages and is subject to certain constraints, some of which are associated with shaft construction.

INTRODUCTION

United Kingdom Nirex Limited (Nirex) was set up by the nuclear industry in 1982 with the agreement of the Government, to carry out the national strategy for the disposal of solid low and intermediate level radioactive wastes. The Company is proposing to dispose of both types of waste in vaults excavated deep underground either on the UK land mass or beneath the seabed.

Initial design work undertaken by Nirex was aimed at identifying feasible repository concepts and the results of this work were outlined in the Nirex publication 'The Way Forward'.[1] More recently these concepts have been adapted to a variety of host rocks and settings in order to identify the costs and constraints of developing such facilities.

In late 1987 UK Nirex Ltd commissioned 2 design studies for the development of a national repository for deep underground disposal of intermediate and low level radioactive waste. A study of a repository accessed from a land base was performed by the Costain-Arup-Electrowatt Consortium, supported by James Williamsons Associates and AMCO. A design team headed by Sir Robert McAlpine and Sons Limited performed a similar study for an offshore repository on the UK Continental Shelf.

The geological environments included in the land-based design work were: a hard crystalline rock in low relief terrain, seaward dipping clastic and non-clastic sedimentary rocks and basement rocks under sedimentary cover (BUSC). Design concepts were developed for a number of typical repository settings. These comprised inland, coastal and small island sites with disposal vaults either under land or beneath the seabed but accessed from land. A repository at British Nuclear Fuels Sellafield site, the source of much of the UK's radioactive waste, was also considered. Two different host rocks were assumed for this site specific case; a relatively deep seaward dipping stratum of anhydrite and a shallower stratum of hard basement rock underlying sedimentary cover.

The offshore repository design work was based on a typical hard rock site to the West of the UK landmass and a repository in chalk or a salt dome intrusion in chalk off the East coast of England.

In carrying out the design work the consultants were required to prepare a number of conceptual repository designs for each of these basic situations and thereby identify and quantify the costs and geotechnical constraints associated with the potential settings and host rocks.

In addition, an appraisal of the safety of the repository during the construction, operational, backfilling and closure phases was required as part of the design study. Both conventional and radiological safety aspects were considered. Nirex provided guidance on the radiological protection standards to be adopted in accordance with regulatory requirements and the principles established by the regulatory authorities.

Data on the geology, hydrogeology and geotechnology relevant to each of the host environments were provided by Nirex. This information was based largely on reports commissioned from the British Geological Survey.

It is planned for the deep repository to receive its first waste in the early years of the next century and for it to be operational for a period of about 50 years. During this period it is estimated that up to 2.1×10^6 m^3 of radioactive waste, comprising 0.6×10^6 m^3 of intermediate level waste (ILW) and 1.5×106 m3 of low level waste (LLW), will be sent to the repository. These volumes are based on a projection of the 1986 DoE/Nirex Inventory[2] and are those of conditioned waste, ie after volume reduction and immobilisation, but excluding packaging and shielding.

The purpose of this paper is to summarise Nirex's conceptual design schemes for a deep repository and in particular to describe possible means of vault access considered by the designers.

DEEP REPOSITORY VAULTS ACCESSED FROM A LAND BASE

Disposal vaults in hard rocks

The study assumed that waste disposal in a hard crystalline rock would be at 500 m depth in a strong, unfaulted granitic intrusion. Large cavern vaults of up to 25 m wide×35 m high× 250 m long were chosen to minimise the volume and cost of excavation and maximise packing efficiency of the waste.

Twenty-six caverns would be laid out on a single level grid pattern with parallel vaults on either side of a group of spine tunnels (Fig 1) which would be used for waste delivery, maintenance, services and active ventilation extract. Two levels of tunnel around the perimeter of the vault area (about 85 Ha) would provide access for construction, backfilling and ventilation air supply. Cavern construction would proceed in parallel with waste emplacement, but construction and emplacement operations and ventilation would be kept physically separate.

A conceptual design was also prepared for a repository in a hard anhydrite deposit which is believed to exist near Sellafield. The anhydrite stratum at the representative site was assumed to be about 60m thick and 1300 m depth, to be overlain by sandstone and to dip seaward at a gradient of between 1 in 6 and 1 in 4.

Eighty-two 16m wide×20 m high×250 m long cavern vaults were proposed for waste disposal. The overall layout would be similar to that for the 'hard rock' concept, although 3 sets of spine and perimeter tunnels would be used to optimise the arrangement as shown in Fig 2.

This design would present a number of construction and operational difficulties because of the very deep vaults, the dipping stratum, the presence of an overlying sandstone aquifer, the relatively high ambient temperature at vault depth (50°C) and the expansive reaction of anhydrite if it came in contact with water. The high underground temperatures and sensitivity of anhydrite to water would require careful control of air conditioning, ventilation and drainage.

The implications of locating the repository on an island site rather than the mainland were also examined. A 'hard crystalline rock' host geology was chosen for this exercise although other host rocks could have been chosen since the underground design was not sensitive to the island setting.

Disposal vaults in soft sedimentary rocks

The geology of the representative site considered in the study comprises chalk overlying Jurassic clays. Concrete lined tunnels are the only practicable configuration for underground excavations in clays whereas unlined cavern construction is possible in chalk. This constraint, coupled with the fact that less cover is required for LLW than ILW, led to a two-tier design with LLW disposed of in chalk at 100m depth and ILW in clay or mudstone at 300m. Sixty-four caverns in chalk up to 15m wide×18m high, but otherwise similar to those in hard rock, would be required for LLW disposal.

The construction of tunnels in clay for the disposal of ILW is subject to major uncertainties because there is a lack of experience of tunnelling in soft rocks at such depths. However, given that these doubts can be resolved, the underground works would comprise an array of 5 m ID tunnels bifurcating on either side of 3×8 m ID spine tunnels (Fig 3).

Disposal vaults at a BUSC site

The designers did not specifically consider a repository excavated in basement rock underlying sedimentary rock cover. However, such a repository would combine elements of the hard rock and sedimentary rock designs. The vault design and underground layout were assumed to be the same as for the hard rock repository, whereas the access shafts would be similar to those designed for the repository in soft sedimentary rock.

Disposal vaults beneath the seabed accessed from a land base

The designers considered the implications of excavating disposal vaults beneath the seabed for each of the representative sites where these were in a coastal setting. The only significant difference between siting disposal vaults beneath the seabed and beneath land is that the increased length of the access tunnels from the surface receipt facilities to the disposal vaults would add both time and costs to the construction programme and marginally complicate repository operation and ventilation.

Surface facilities

The design of the surface facilities would be similar for all sites and a typical layout is shown in Fig 4. The study considered the relative merits of siting the waste preparation and packaging facilities at the surface or underground. It was concluded that these facilities should be sited at the surface within a single Waste Receipt Building. The surface facilities, covering an area of about 65 Ha, would thus include buffer storage areas for incoming waste, the Waste Receipt Building, shaft headworks/drift access, a contractors' area, active effluent treatment plant, services and administration and personnel facilities.

Waste would arrive at the repository by road or rail. Offsite transport would enter the Waste Receipt Building where waste packages would be off-loaded, checked and prepared for disposal. The packages would then be loaded onto rail mounted Vault Transfer Vehicles and these would be hauled by a locomotive from the Waste Receipt Building to the shaft head through a shallow, shielded tunnel. Each Vault Transfer Vehicle would be uncoupled at the shaft head and loaded into the shaft cage. At the shaft bottom, it would be coupled to an electrically driven underground locomotive and transferred along a tunnel to the vaults.

Although waste packages would be lightly shielded during transfer to the vaults, personnel access to the waste transfer route would not normally be possible and all operations would have to be carried out remotely. However, access on a limited basis to recover from accident or breakdown situations would be possible.

Access requirements—hard rock and BUSC sites

Waste Route

The largest and heaviest packages of waste would be 4 m×2.4 m 1.8 m high, weighing 65 tonnes. These figures result from the design of packages for the transport by rail of decommissioning wastes where there is strong economic incentive to use the largest possible packages.

The hoist design was therefore based on a cage payload of 65 tonnes for the package and 15 tonnes for the Vault Transport Vehicle, plus a cage weight of 20 tonnes, giving an all-up weight of 100 tonnes. Although this is greater than any operational shaft design known to the design team, the view was taken that the system could be designed, on the basis of current technology, to UK manriding safety standards. It was assumed that it may be necessary to bring these packages back to the surface thus the hoisting system should be capable of both lowering and raising the design loads.

By using the cage to its capacity, the number of waste loads could be reduced to 120 per week (average) and 180 per week for the busiest periods of waste handling. The cycle time would probably be relatively long, particularly because of the need for care during loading and unloading the cage but also to limit hoisting speed.

At the hard rock and BUSC sites, the requisite number of loads could be lowered down one 8m internal diameter shaft, working 1 extended or 2 normal shifts per day, but at the Sellafield anhydrite site, 2 shafts would be needed to meet the demand. However, in order to cover outage periods, 2 shafts which were capable of handling waste loads would be provided irrespective of site.

Service and Construction

The site was designed on the basis of a rolling programme of excavation, construction, waste disposal, and backfilling around the wastes. The excavation and construction programme could be quite adequately achieved with 2 dedicated shafts, each having an internal diameter of 6m. One would be used for muck hoisting and the other for service traffic.

Ventilation requirements

The ventilation demand was based on the use of diesel powered plant for hard rock excavation and an additional requirement to control the environment in the waste disposal vaults. This demand would require the provision of 4 shafts of the proposed internal diameters which is in line with access requirements. The shafts would be allocated as follows:

– waste traffic and active ventilation upcast;
– waste traffic and ventilation downcast;
– muck hoisting and ventilation upcast from the excavation workings;
– service traffic and ventilation downcast.

Access requirements—sedimentary rock site

At the sedimentary rock site, 3 shafts of 8m internal diameter would be provided for access to the tunnels in clay/mudstone. One would be dedicated to excavation and construction. The use made of the other 2 would alternate between, waste traffic/active ventilation upcast and backfill supply/ventilation downcast, depending on which of the 2 groups of operational tunnels either side of the spine tunnels was being used for waste disposal and which was being backfilled.

For the caverns in chalk at 100m depth, the design team chose to use 3 drifts at 1:10 gradient rather than shafts for access. The reasons for this are discussed later in the paper. The drift allocation was:

– a drift for waste traffic, for which it is fitted with a rack-and-pinion track, and active ventilation upcast;
– a drift for muck conveyance and ventilation upcast from the excavation workings;
– a drift for service traffic and ventilation downcast.

Access construction

Access to hard crystalline rock vaults

The method of constructing shafts to a depth of 500m in hard crystalline rock would be entirely conventional with drill and blast techniques probably being used for excavation. Groundwater ingress during construction would be controlled by pumping, if possible, or by sheet or bored piling in the foreshaft with grouting or ground freezing techniques used at depth. The need for, or extent of freezing and grouting cannot be determined until site conditions are clearly established. It is assumed that shafts would be lined throughout to withstand the full hydrostatic pressure of the groundwater and that a permanent sump would be provided at the shaft bottom to accommodate cage overrun, pumping equipment, balance rope sag and, in the case of waste delivery shafts, a shock absorber system. A friction winder, supported by a headframe over the shaft and with the cage balanced by a counterweight, would be used for hoisting. It was estimated that the first shaft could be in service 3 years after the start of cons truction with all 4 shafts in service after about 4 years.

Access to anhydrite vaults at Sellafield

The predicted depth of the anhydrite (1300m) and the need to provide access through the overlying water bearing sandstone would make shaft construction at this site particularly challenging. The groundwater may be saline which would add to the difficulty of using freezing techniques which in any case are likely to be useful only for the top 600 m. In addition, the depth would affect shaft operation and 2 shafts, dedicated to waste delivery, would be necessary to achieve the necessary throughput.

The shaft lining would have to be designed against the full hydrostatic head of water from the surface down to a grout seal, probably in the St. Bees shales (1000 m deep). It is therefore likely that at depth a composite lining of reinforced concrete and ductile cast iron or carbon steel tubbing would be needed. Excavation would probably again be by the drill and blast method but the rate of shaft sinking would be slowed down by the need to control groundwater. This, together with the increased depth, was estimated to increase the time required to construct the first shaft to $6\frac{1}{2}$ years with the fourth completed after $7\frac{1}{2}$ years. The rate of shaft construction can only be broadly estimated and possible lower rates could significantly lengthen the repository construction programme.

Access to soft sedimentary rock vaults

Where the vaults are sited at a distance from the surface installation equal to or greater than 10 times the depth, drift access might be cheaper than the equivalent shafts plus tunnels. Drift access was therefore considered by the designers for access to the shallower LLW vaults in chalk although this decision would clearly have to be reconsidered at a specific site. Three drifts 8 m wide×6 m high with a curved roof would be required, but a circular drift profile might have to be used in deep water bearing strata or in weak rocks. The presence of aquifers in the chalk would require special measures to control groundwater inflow during excavation and cement grouting was proposed. The lining would need to resist hydrostatic pressure around the entire perimeter as well as pressure from the surrounding rock.

Shaft sinking in comparatively soft sedimentary rock is in principle similar to sinking in hard rock. However, it is likely that the foreshaft would have to be sheet piled through a substantial thickness of boulder clay overlying the chalk. The shaft lining would need to be designed to resist full hydrostatic head and freezing or cement grouting would be required to control groundwater inflow during excavation in chalk. Although the stratum for ILW tunnels is described as Jurassic clay, it may be an over consolidated clay or a mudstone. More confidence was expressed in constructing insets at 300m depth in mudstone than in clay and it was recommended that, once site specific information was available, the feasibility of constructing these and the tunnel junctions should be reassessed.

It was estimated that the first shaft would be complete 2½ years after the start of construction with all 3 shafts completed after about 3 years.

Access to vaults in BUSC

Shaft sinking at a BUSC site would be through a sedimentary sequence and would thus be similar in concept to the ILW shafts at a sedimentary rock site.

REPOSITORIES ACCESSED FROM AN OFFSHORE STRUCTURE[3]

Reference sites

The design study for an offshore repository was undertaken by Sir Robert McAlpine and Sons in association with NEI Limited and James Williamsons Associates. Three reference locations, all with a water depth of 60 m, a minimum strata thickness of 1500 m and seabed conditions suitable for founding an offshore structure, were considered for an offshore repository:

a) in high grade metamorphic and igneous rocks outcropping off the West coast of Scotland;
b) in chalk off the East coast of England;
c) in a salt dome intrusion in chalk off the East coast of England.

These were selected primarily on the basis of geological characteristics but took account of other factors such as shipping lanes, environmental impact, water depth, military use, and proximity to exploitable mineral resources. Two vault designs were considered, based on the results of earlier feasibility studies:

- caverns at 300m depth, with access from shafts excavated from offshore structures;
- large diameter disposal shafts up to 1300m deep for direct placement of waste from an offshore structure.

Both vault designs were found to be satisfactory for the hard rock site. However, disposal shafts in chalk would require expensive temporary support and permanent shaft lining throughout, thereby limiting the practical depth to which they could be constructed, and these were therefore considered to be uneconomic. The creep characteristics of rock salt and the consequences of failure of the watertight lining of one of the multiple penetrations of the cap rock seal, leading to the release of radionuclides through leaching, would make both caverns and disposal shafts in this geology and setting impracticable.

Caverns

The underground layout and operation of the offshore cavern repository (Fig 5) would be similar in most respects to the onshore repository. Access to the repository would be provided by five 8 m internal diameter shafts excavated from concrete gravity platforms founded on the seabed. One platform would be dedicated to hoisting radioactive wastes and the exhaust of foul air from disposal operations. The other would be dedicated to all non-nuclear activities including the supply of fresh air, the handling of all backfill materials and the extraction of spoil and exhaust air arising from the construction stages.

Large diameter disposal shafts

The study concluded that large diameter disposal shafts in hard rock did not need lining, other than through the seabed interface, and should be 13 to 15 m internal diameter and 1000 to 1300 m deep for economic construction and efficient packing. Groups of 7 disposal shafts would be sunk from a hexagonal concrete gravity platform founded on the seabed, similar units being joined together and connected to a single waste receipt platform to form a large artificial island Fig 6.

Surface facilities

A port facility would be a common feature of both repository designs and a typical layout is shown in Fig 7. Its function would be to receive and provide buffer storage for incoming waste and construction materials and to load them onto vessels for transfer to the offshore locations.

Waste would arrive at the port facility by road, rail or sea. There the waste packages, within transport shielding where applicable, would be loaded into large cellular handling crates, called ‘megacrates’, which would contain up to 48 packages and weigh up to 2000 tonnes. These would be transported offshore ons a ‘Semi-Submersible Crane Vessel’. On the offshore platform the packages would be removed from the megacrates and transferred to a shielded handling cell adjacent to the disposal vault where they would be checked and prepared for disposal.

In the case of the cavern repository a shielded cell would be situated underground at the end of each cavern. Waste packages would be transferred to the caverns via the waste delivery shaft in a similar way to the onshore concept.

In the case of the disposal shaft repository the shielded handling cell would be situated at the head of the shaft and be transferable between shafts. Here waste packages would be assembled on a ‘transfer bogie’ in the same positions relative to each other in plan that they would occupy in the shaft. The bogie would be moved over the shaft, a lifting frame would lift the packages from the bogie, which would then be withdrawn, and the assembly (65t maximum payload) lowered down the shaft. This operation would be repeated until a layer of packages was complete. Backfill would be placed by skip after the completion of each layer. Five disposal shafts, containing different waste package types, would be operational at any time and all the operations described would be carried out remotely.

Access Requirements

Drift access offshore was not considered because a much larger surface structure would be required to accommodate such drifts down to seabed level and the use of drifts would lengthen the access route through the seabed sediments.

A cavern repository would be accessed from 2 concrete gravity platforms. The construction platform would be fitted with 3 shafts, 8m internal diameter, used as follows:

- a rockshaft for spoil removal and construction ventilation upcast;
- a construction access shaft for construction personnel and plant access and ventilation downcast;
- a backfill shaft for backfill supply and ventilation downcast.

The operation platform would be fitted with 2 shafts, 8m internal diameter, used as follows:

- a waste delivery shaft, not used for ventilation;
- an operational access shaft for operational personnel and plant access and ventilation upcast.

Waste throughput of up to 320 disposal packages and 130 returning transport containers per week would be achieved by operating 2 cages in the waste delivery shaft.

A disposal shaft repository would require about 34×15 m internal diameter×1300 m deep shafts for waste, plus 1 common 6 m internal diameter rockshaft for spoil removal, all contained within an offshore structure comprising 5 hexagonal units. The top 200m of each shaft would not be used for waste disposal, but would be backfilled to provide a seal. Waste throughput would be achieved by operating up to 5 shafts simultaneously with a rolling programme of shaft construction proceeding in parallel with operations.

Shaft construction—cavern repository

A number of conventional methods could be used in constructing shafts, including sinking, slashing, blind drilling, open boring or raiseboring. The choice would depend on the ground conditions at the chosen site and future developments in construction techniques. In-situ concrete linings, which would be unreinforced except at insets and above rockhead, were thought to be the most economic method of excluding groundwater and providing structural support. The choice of geological environment offshore should ensure that no aquifers were encountered below the fractured weathered zone, assumed to extend to a maximum of 50m below seabed, and any water flows in this area would be treated by grouting. It is doubtful that freezing would be effective in igneous and high grade metamorphic rocks. Freezing would be effective in chalk but in such cases the shaft lining would be designed for full hydrostatic pressure. Considerable back-grouting would be necessary behind the lining to ensure that no direct path remained between the waste and the seabed for the passage of radionuclides by leaching.

The shaft/seabed interface is an area of particular interest and the stages of construction are illustrated in Fig 8. The method of sealing at the interface would rely on the construction of a watertight shaft lining extending from the top of the offshore structure, through the overburden and weathered rock into sound rock below the fractured, weathered zone. The upper part of the shaft would probably be constructed within a cofferdam sunk from the offshore structure to give access to rockhead. This could be sealed by a pattern of holes for injecting cementitious and/or chemical grouts. Construction of the foreshaft would then follow using normal blind shaft sinking techniques, the rock being further grouted as required.

The unlikely event of failure of the shaft/seabed seal due to seismic disturbance or abnormal ship collision was addressed by the designers. The design of the shaft through the shaft/seabed interface would allow for loading induced by ‘credible’ ship collisions and ‘reference’ seismic disturbances. They also recommended the installation of sector gates, at the head of each shaft or in the tunnels at the pit bottom, which would shut off and seal the underground workings. In the event of severe damage to the shaft lining, leading to a potential flooding risk, the sector gates would be closed to protect the considerable investment underground and ensure the escape of personnel through another shaft. The seal provided by the sector gates should reduce water inflow to pumpable levels. The designers did not foresee any catastrophic event which would lead to sudden inundation.

Construction of large diameter disposal shafts

Disposal shafts should be of the maximum possible diameter for the best packing efficiency and an economical depth would be between 1000 m and 1300 m, taking account of packing efficiency and construction costs. The diameter should remain constant throughout the depth of the shaft to facilitate waste handling. Information on the feasibility and cost of deep, large diameter shafts is very limited and largely site specific. It should be noted that shaft construction would require a combination of technologies which, although proven in isolation, have never before been brought together on the scale required for a disposal shaft repository. For example, in the UK deep shafts of up to 10m finished diameter have been constructed onshore, as have very much greater diameter but very much shallower shafts. Also, offshore platform technology is proven. However, the construction of shafts of 15m finished diameter to depths greater than 1000m in an offshore environment is without precedent. Nevertheless, given the right geological, hydrogeological and geotechnical conditions, the designers consider that construction of shafts of the required maximum diameter and depth should be feasible.

The minimum spacing between shafts would be 3 diameters between centres based on stress considerations. However, there is little experience in sinking deep shafts in groups of more than 2 and the spacing of an array of shafts may be controlled more by the effects of the pre-treatment (ground freezing etc) required to negotiate the seabed sediments.

Construction would commence with the sinking of a rockshaft by conventional means with the shaft/seabed interface constructed in a similar fashion to that illustrated for a cavern repository shaft. A series of tunnels would connect the rockshaft bottom to the bottom of each disposal shaft. Disposal shafts would be constructed by first sinking and lining a full diameter foreshaft through the sediments and fissured rock below the rockhead within a cofferdam, following the same procedure described for other shafts. A 300mm diameter pilot hole would be bored to the full depth, possibly using remote steering and surveying techniques currently under development, and this would subsequently be opened out to 2 or 3 m diameter by raiseboring. Finally, the full diameter would be excavated by shaft slashing. Shafts in hard rock would be unlined, other than through the seabed interface, and unsupported except for rockbolts and shotcrete.

The possible failure of the shaft/seabed seal was again addressed by the designers, but it was considered that in this case it would be impractical to protect each disposal shaft with sector gates and the pit bottom area would therefore be at risk from flooding.

CONCLUSIONS

A wide range of geological environments and settings has been considered for siting a deep repository and there is little doubt that such a repository could be built at any of the representative sites. However, the various approaches are subject to a number of constraints and some of those associated with shaft construction have been identified in this paper. Whereas shaft construction in hard rock would be largely straightforward, sinking deep shafts at Sellafield through the thick sandstone aquifer to the underlying anhydrite deposit would be challenging. In the case of a sub-seabed repository, excavated from an offshore platform, the integrity of the shaft/seabed interface would be of particular significance. There is also little experience of sinking deep, large diameter shafts in an offshore environment.

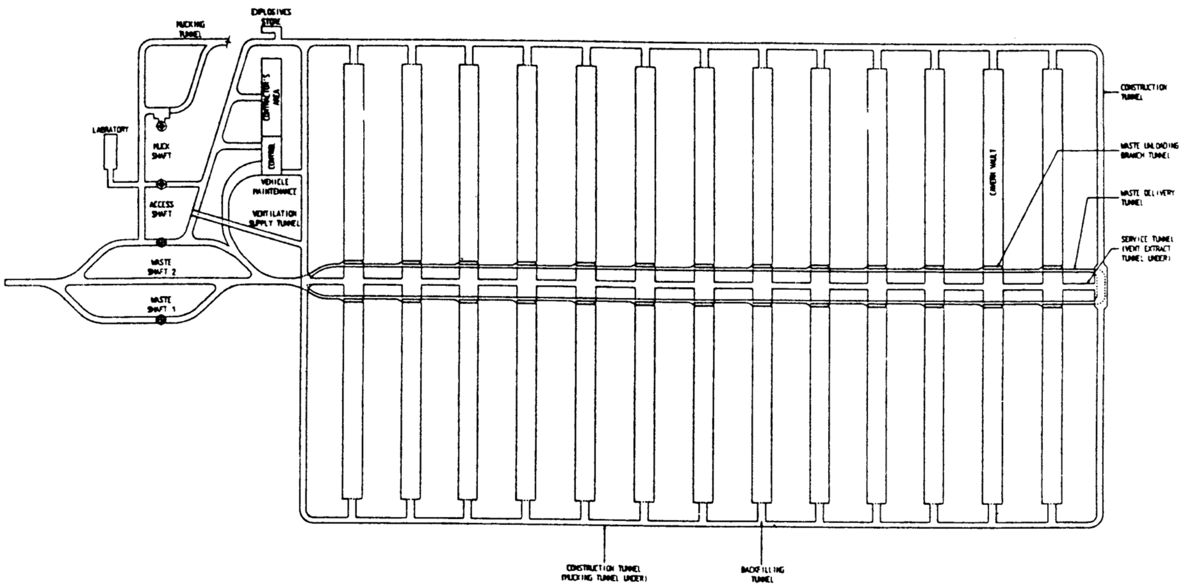

Fig. 1 Land Based Cavern Repository

REFERENCES

1. UK Nirex Lted. ‘The Way Forward. A Discussion Document’. November 1987.
2. UK Nirex Ltd Report Number 43. ‘The 1986 United Kingdom Radioactive Waste Inventory’. DoE/RW/87 088.
3. Beale H, UK Nirex Ltd, Taylor S J, Sir Robert McAlpine and Sons Limited. ‘Deep Repository Design—Offshore Concepts’. Society for Underwater Technology, Disposal of Radioactive Wastes in Seabed Sediments, International Conference, St Catherine’s College, Oxford. 20/21 September 1988.

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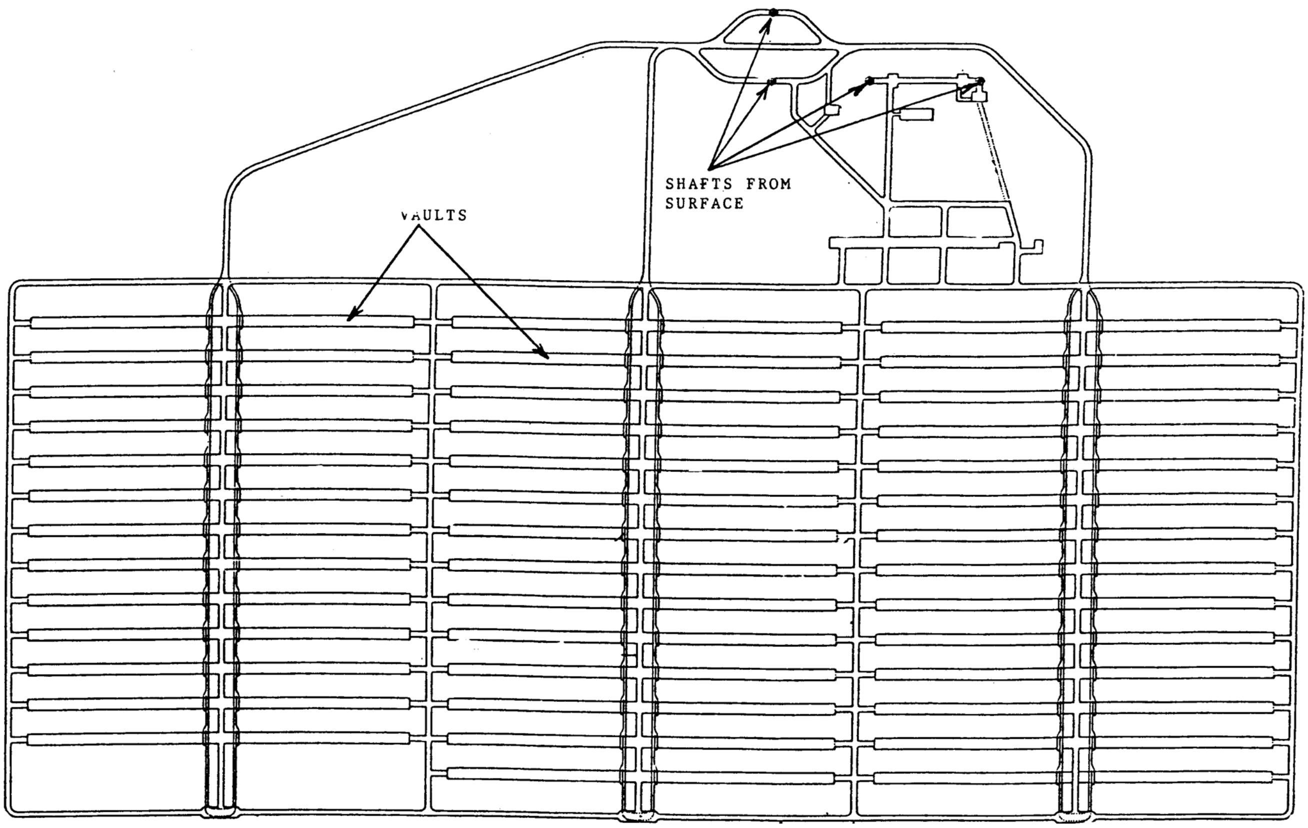

Fig. 2 Sellafield Repository: Outline Design Showing Vault Layout

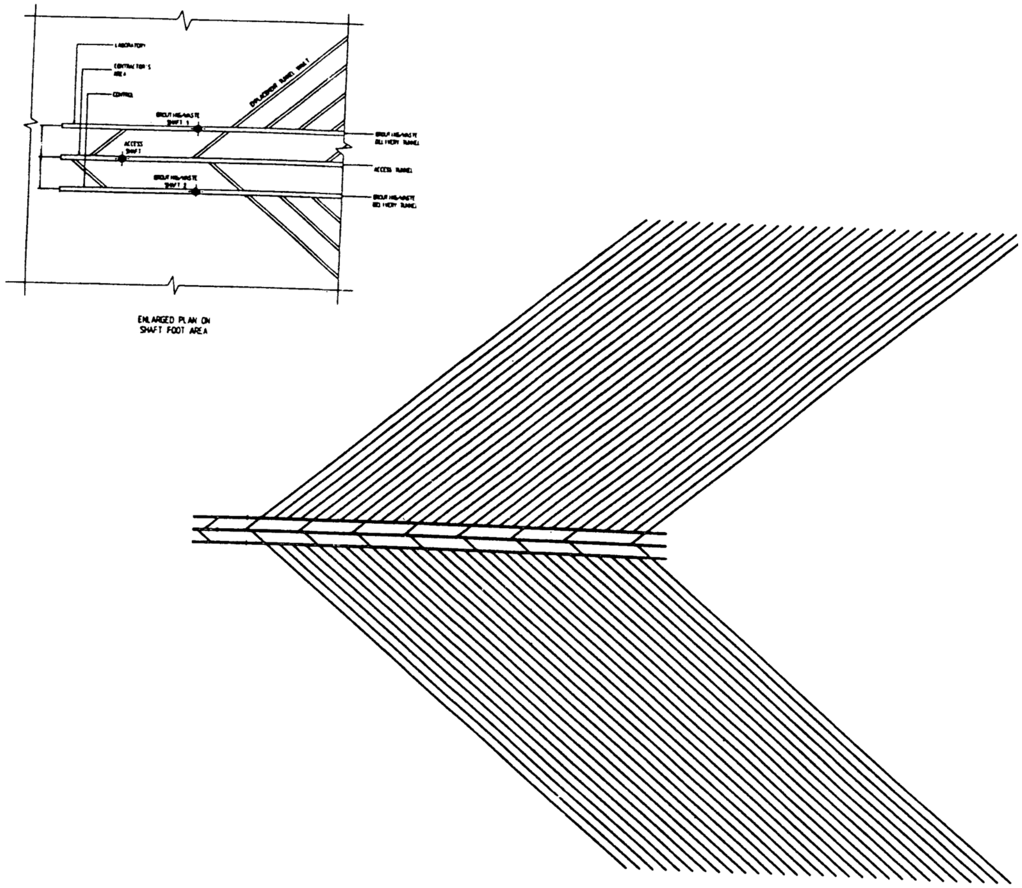

Fig. 3 Land Based Tunnel Repository

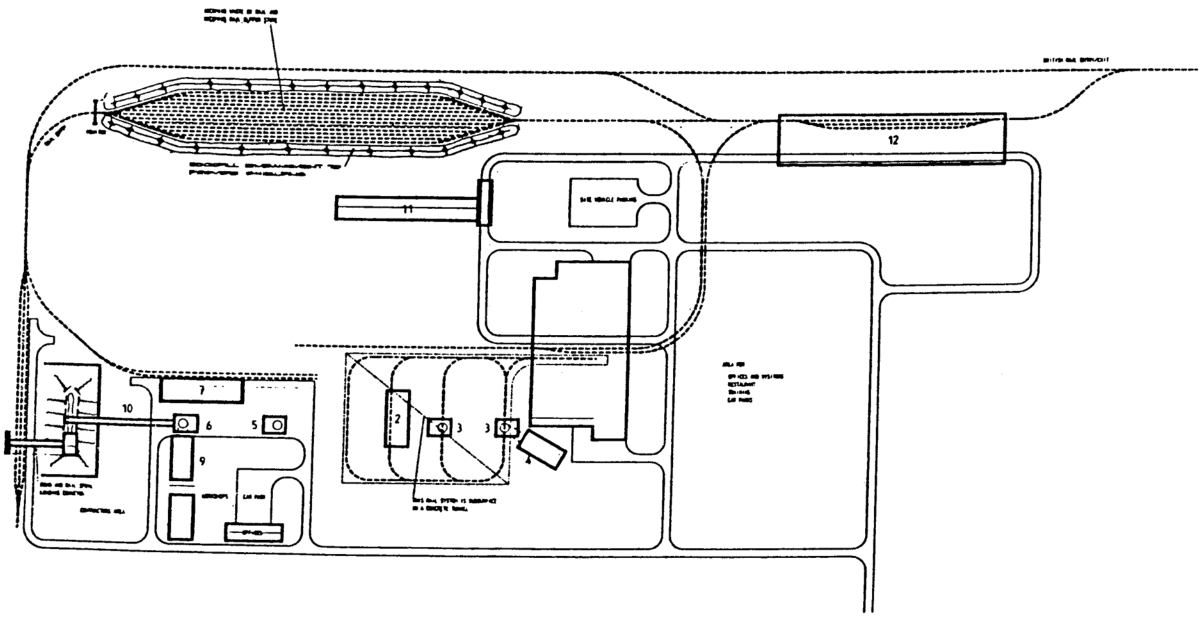

Fig. 4 Proposed Layout of Surface Facilities for a Land Based Repository

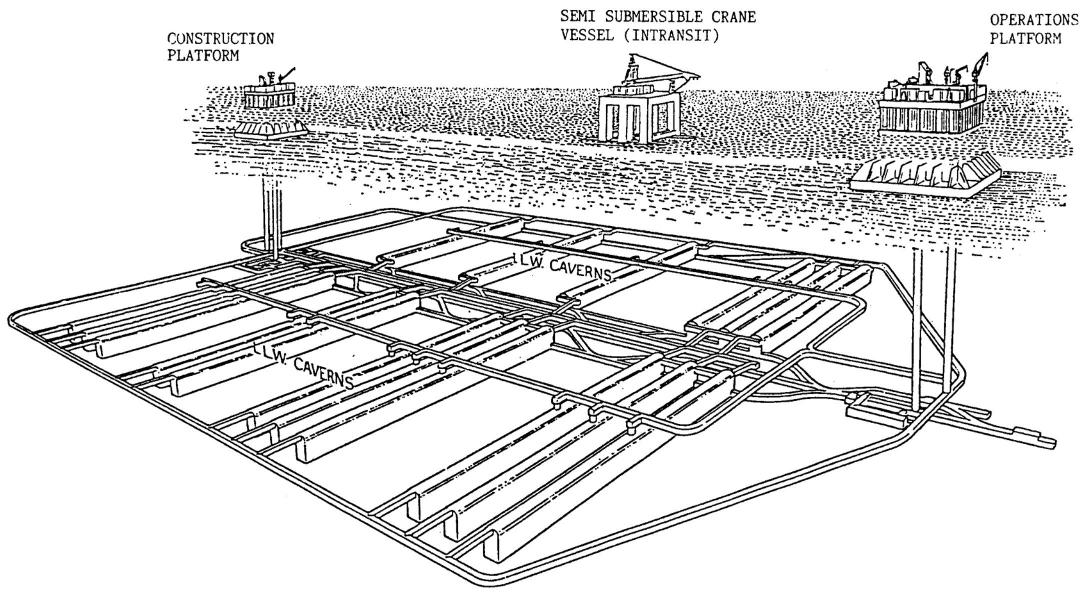

Fig. 5 Cavern Repository Accessed from Offshore Structures

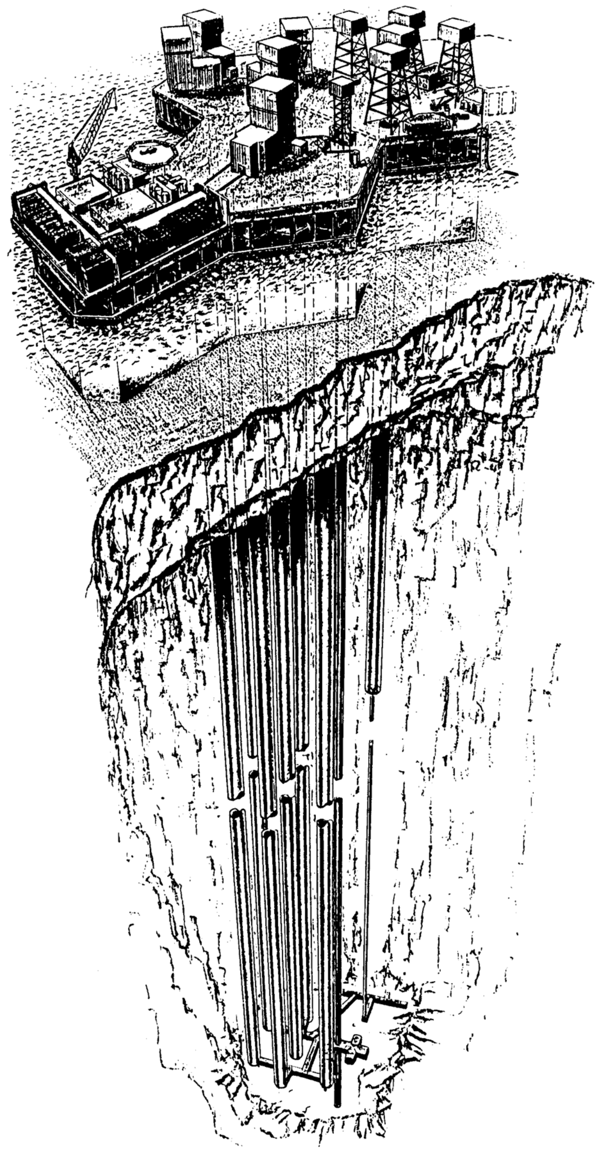

Fig. 6 Offshore Borehole Repository

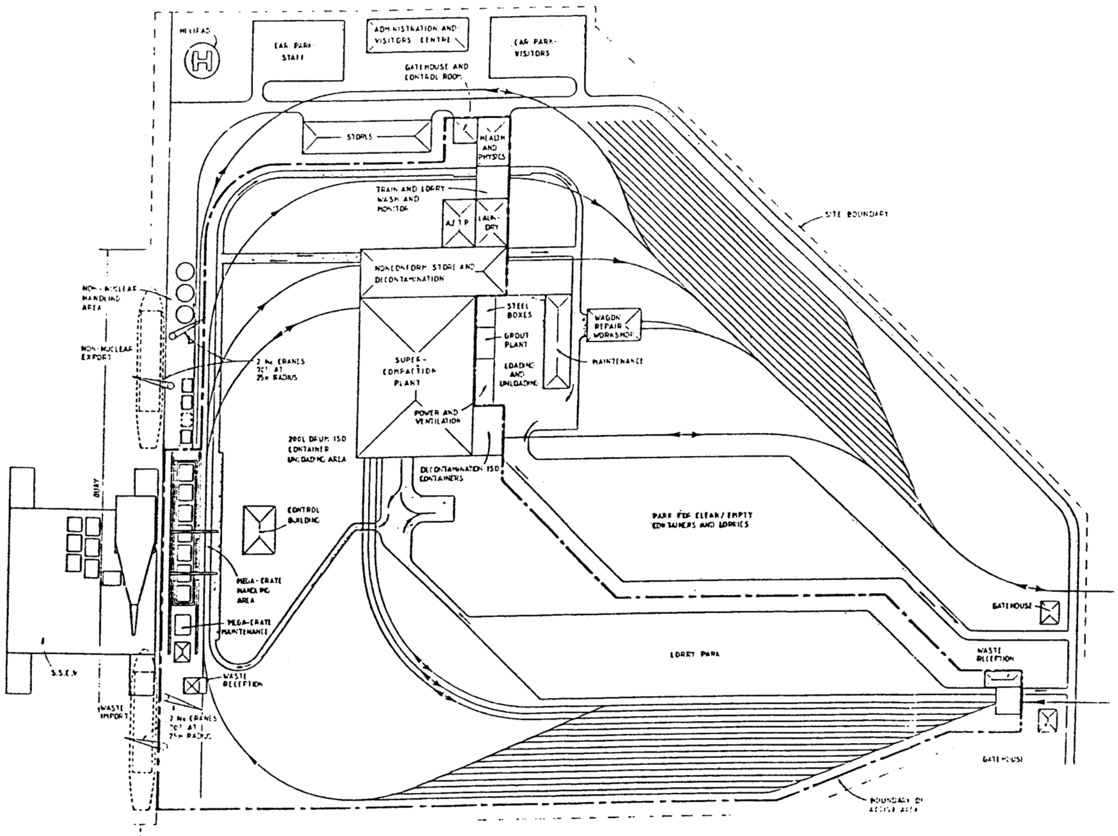

Fig. 7 Proposed Layout of Disposal Port

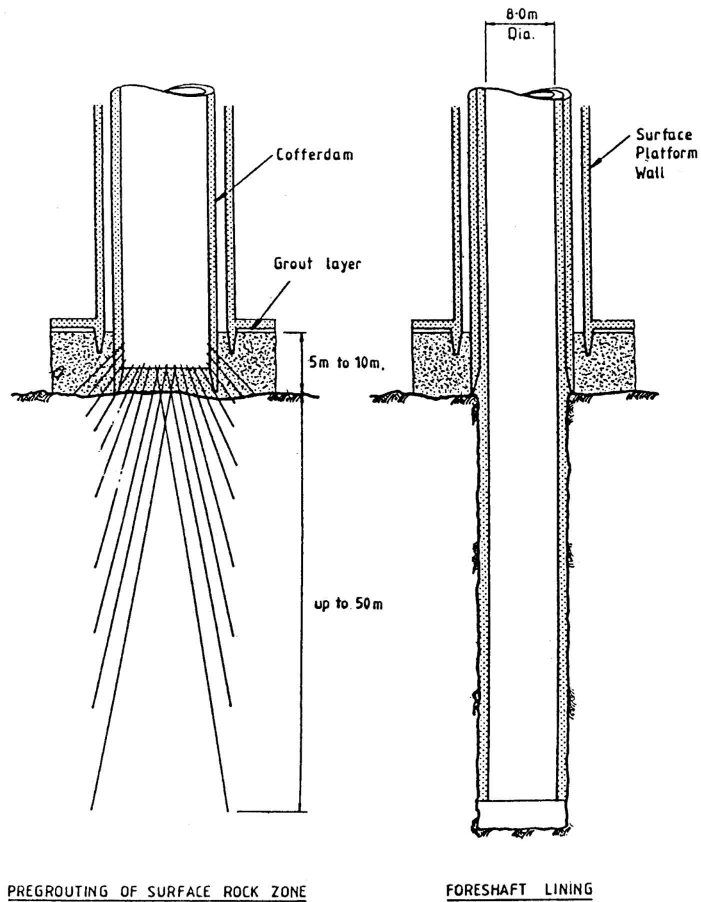

Fig. 8 Seabed Interface Stages of Construction

An approach to field testing and design for deep mine shafts in the Western U.S.A.

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Figure 1. Map showing location of Coeur d'Alene Mining District and the Homestake Mine in the northwestern United States.

SYNOPSIS

Deep mine shafts in Idaho and South Dakota were instrumented to evaluate performance of the shafts during construction and after the shaft was in service. Objectives of the work are to develop design guidelines for circular and rectangular-shaped deep shafts and determine effects of nearby mining on shaft stability. In addi-tion, data are being used in real-time to evaluate shaft performance during construction and to serve as a warning of impending Instability after the shaft is in service. Field testing was conducted at maximum depths of 2,000 m below the surface in hard, brittle rock subject to high, nonuniform stress conditions. The high stress level results in excessive deformation of the shaft opening and failure conditions. Guide misalignment and recurring shaft repair and maintenance are thus a major problem.

The testing approach is to measure rock mass displacement, stress, and strain in the concrete lining and axial loads on shaft timber and steel sets and rock bolts. Data are collected and analyzed by a computerized system in the mine and downloaded via a modem to the Bureau of Mines Research Center in Spokane, Washington. Detailed analyses Include time history and face advance plots and nonlinear regression analysis to establish simple behavioral relationships.

Design approaches include closed-form solutions as well as finite-difference modeling. By comparing measured data to calculated values, an appropriate numerical model was developed and shaft design criteria, such as lining thickness and support delay, were established.

INTRODUCTION

A shaft is truly the “lifeline” of an underground mine. Damage to the shaft lining and guides as a result of ground movement can result in serious loss of production and extensive and continual repair. In some districts, annual shaft repair and maintenance costs resulting from high rock stresses and excessive displacement are in the millions of dollars a year.

For many years, the Bureau of Mines has conducted research to characterize the rock mass and to develop structural guidelines to better design shafts in deep mines. Much of this work has centered on determining the in situ conditions that affect the structural stability of shafts in the Coeur d'Alene Mining District of northern Idaho and the Homestake Mine near Lead, South Dakota (Figure 1). Recent work has concentrated on measuring the rock and support behavior during shaft sinking and after the shaft is put into service.[1–3] The results of this work can be used as a comparative data base for further basic research as well as to establish design criteria and provide guidelines for monitoring shaft stability during and after construction.

SHAFT MEASUREMENTS

Preliminary Considerations

A shaft measurement system, including orienting and locating instruments, is based on projected in situ stress conditions, shaft geometry, and geologic environment. A basic instrumentation approach is to obtain the maximum amount of information with minimal Interference with shaft sinking activities. A totally sealed, remotely accessed, environmentally secure package that exposes no electronic components once the system is transported down the shaft is required.

Paramount to designing and Implementing a shaft measurement scheme successfully is recognizing that the installation must be conducted under severe time restraints and environmental conditions. In most cases, interference with shaft sinking activities or hoisting must be held to an absolute minimum. Furthermore, once the installation is complete, it is very difficult, if not impossible, to access the installation for maintenance or repair or to obtain data. Therefore, before any other consideration, Installation must be completed rapidly, it must be extremely reliable, and it must be capable of being remotely monitored. These goals must be accomplished despite the hazardous environmental conditions in a shaft, including (1) limited access, (2) cramped working space, (3) 100% humidity, (6) high water inflow, (5) high rock temperature, (6) shock and flyrock from blasting, and (7) mud, dust, and falling debris.

The response of the rock mass depends on excavation, shaft orientation (both with respect to geology and in situ stresses), and distance between the shaft bottom and the installed support. Sensors must be chosen to collect as much of this information as possible under constraints of time and budget. Measurements indicate the behavior of the rock mass and the effectiveness and safety factor of the ground support system. Radial closure provides valuable data for judging the stability of the shaft. If closure accelerates or falls to stabilize, the shaft wall and concrete lining are in danger of falling.

The effectiveness of a ground support system can be enhanced by increasing rock mass cohesion and the confinement provided at free faces. Cohesion of the rock mass is increased by installation of bolts that intersect planes of weakness and prevent block movement through the application of confining pressure. This pressure can determined by measuring the tension in individual bolts with strain gauges or rock bolt load cells.

The most common permanent shaft support system is a concrete liner, which confines the rock mass and seals the surface to prevent rock degradation and spalling. As do rock bolts, the rock mass must deform for a load to develop in the liner. Concrete linings are relatively brittle and could rupture if overloaded. Sampling of the hoop (tangential) stress and strain yields an estimate of the support load, strength safety factors, concrete mass modulus, and an estimate of the situ stress field.

In rectangular shafts, timber or steel sets are commonly used for a support framework. Measurements of axial loads on a timber set may yield data to evaluate shaft guide alignment problems or problems with severe concentrated side loading caused by nearby mining. Thus, the most important measurements are radial deformation of the shaft, tangential stress and strain in the lining, and loads on timber and steel sets and rock bolts.

Rock Mass Deformation

Deformation is best measured with multiple-position borehole extensometers (MPBX's), using a deep-seated anchor as a zero reference. Positioning the deepest anchor and specifying the range and accuracy of transducers requires predicting radial displacement as a function of distance into the shaft wall and distance to the shaft bottom. Elasticity theory predicts a reduction of radial closure rate with continued shaft advance. Radial deformation also decays rapidly with increasing distance from the shaft wall (Figure 2). The curves represent the total and measurable elastic deformation, which, at the shaft wall, is about 30% of the total. The change in displacement with radial distance approaches zero at about 15 m, or twice an unlined shaft diameter of 7 m. Thus, for purely elastic behavior, an anchor at two diameters is considered stable.

In addition to the deep reference and collar be anchors, other intermediate anchors provide a radial deformation profile of the borehole. The profile is useful for delineating zones of block movement and rock yielding, which are important in designing shaft support, as well as for confirming the stability of the deepest anchor. Figure 3A shows installation of an MPBX in a shaft and typical layouts for ircular and rectangular openings are flustrated in Figures 3Band 3C, respectively. This layout optimizes the the effects of the expected response factors because of shaft geometry and geologic conditions.

Lining Stress and Strain

Measurement of tangential lining pressure is complicated by superimposed local bending and shear stresses and point loads. Shear stresses are avoided using a flat jack pressure cell design, which is relatively insensitive to non-normal loads. The Influence of liner bending stress is minimized by positioning the cells to measure tangential stress at the neutral axis of the lining. Point loading may be minimized by pre-encapsulating the cell in concrete or hydrostone. The tangential load produced

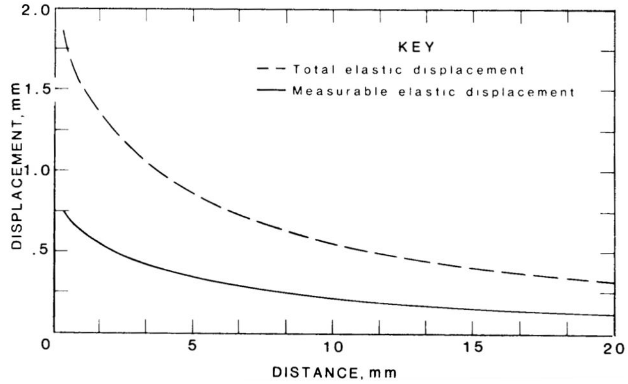

Figure 2. Total and measurable elastic rock displacement as a function of radial distance.

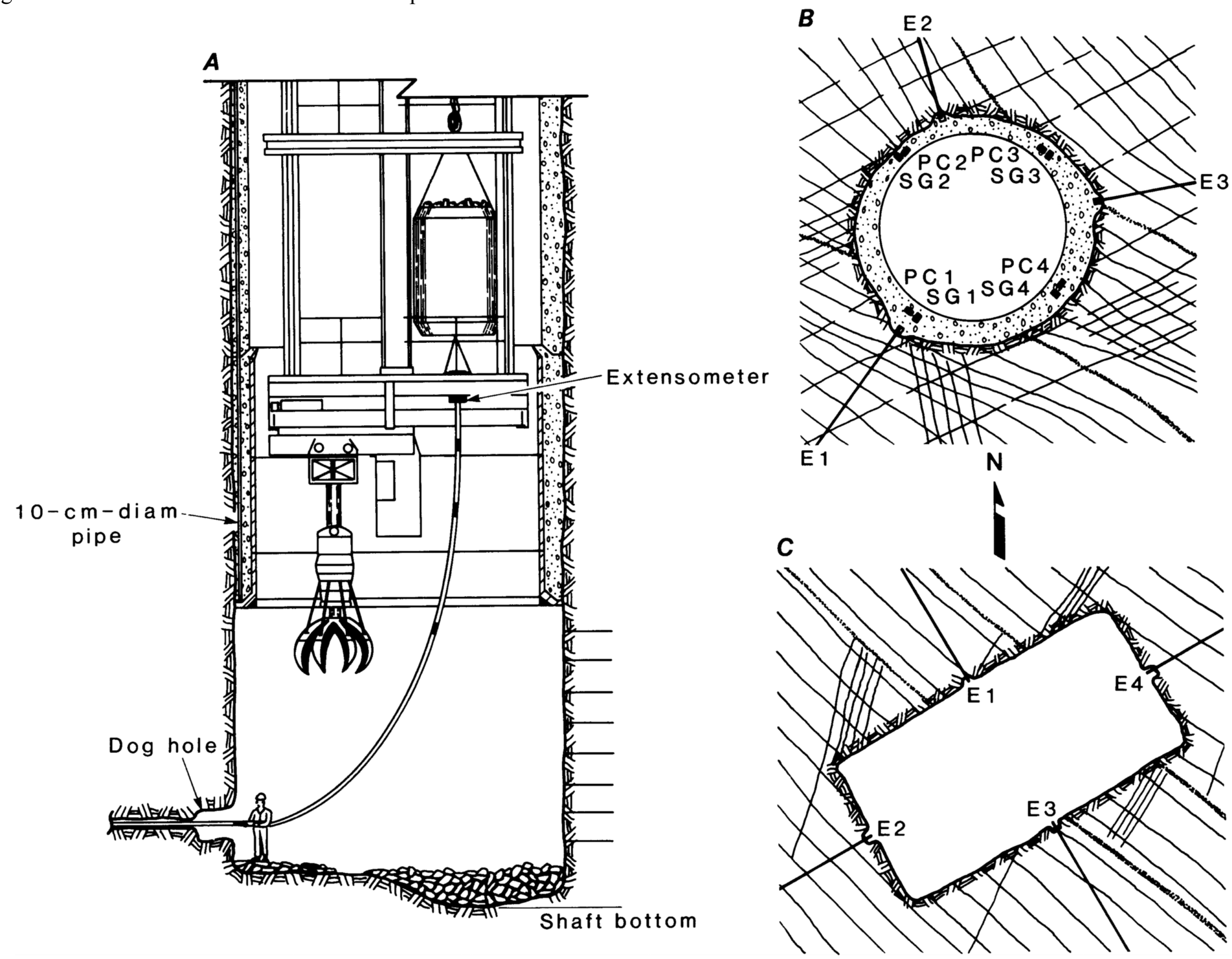

Figure 3. Typical Installation of MPBX in shaft. (A) Vertical section, (B) plan view of circular configuration, and (C) plan view of rectangular configuration.

by a given displace ment is estimated by thick-wall cyl inder solutions for radial displacement and tangential stress from an external hydrostatic pressure. The pressure cells thus correspond to an expected tangential stress of 21 MPa.

Concrete typically displays an increasing strength and deformation modulus with time, suggesting that lining strain is load-path dependent. Early loading of the lining while still green may be relieved by creep of the concrete lining. Finding the amount of creep and the effect of increasing modulus requires measurement of strain in the concrete lining. Concrete embedment strain gauges are used to verify data from tangential pressure cells and to provide an estimate of concrete modulus and in situ

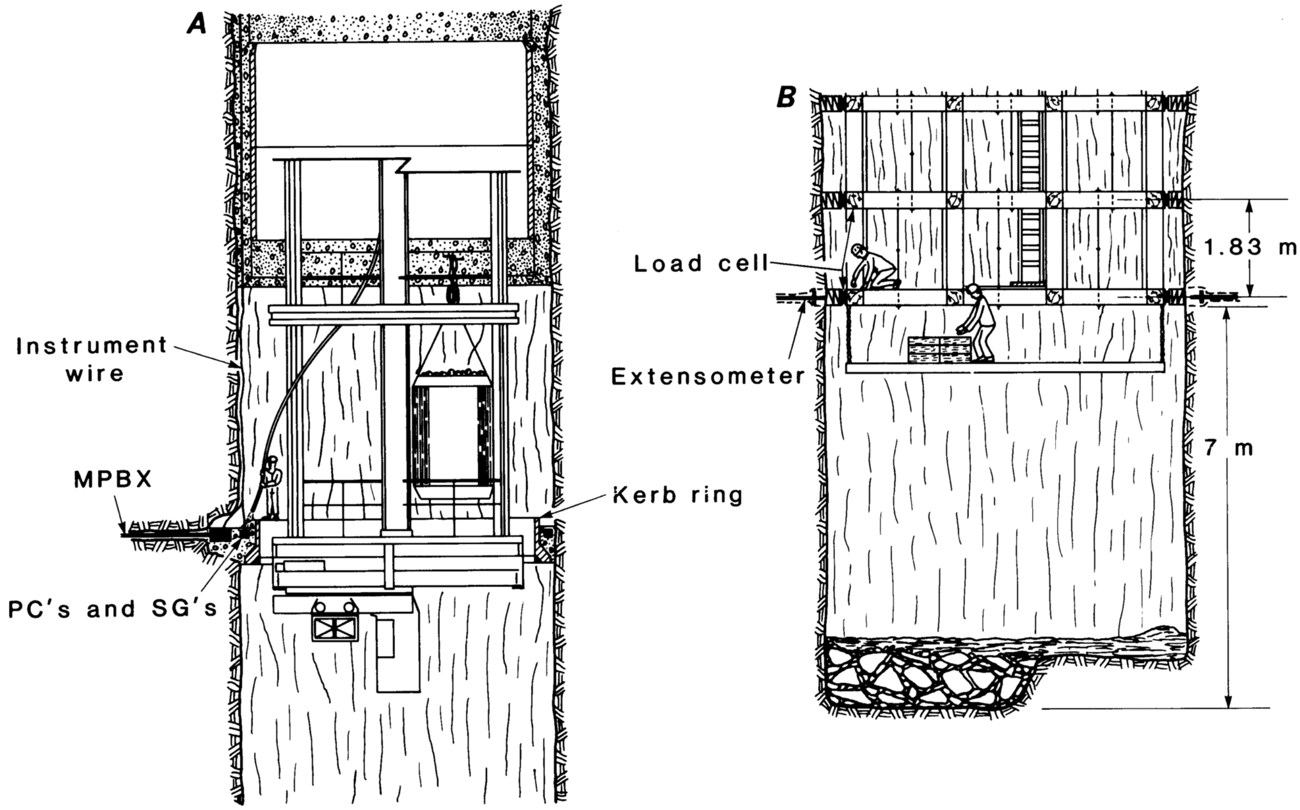

Figure 4. Typical Installation of (A) PC's, (B) SC's in a concrete lining, and (C) flat compression load cells in timber sets in rectangular shaft.

stress field. Figure 4A shows typical installation of pressure cells (PC's) and strain gauges (SG's). They are located to measure tangential pressure in the four quadrants of the liner.

Rock Bolt and Timber Set Loads

The axial loads on mechanically anchored rock bolts and on timber or steel set blocking provide an excellent indicator of shaft stability. Deep-seated rock bolts are, in effect, "sampling" a large volume of shaft wall rock, and displace-ment-induced loading can be an extremely sensitive indicator of pending instability. Measurement of shaft set loads provide warnings of possible guide misalignment problems and localized rock mass failures. Figure 4B shows installation of flat compression load cells (FCLC's) in blocking in a timber set.

Temperature

Curing of the concrete lining, variation in mine ventilation temperatures, and seasonal changes produce significant amounts of heat changes in the instruments after installation. Therefore, sensor temperature must be monitored for correction of instrument data. Thermistors and PRT's are installed to handle a range of temperatures from 60° to 160° C.

Data Collection

The primary concern when collecting data in a shaft is that the data acquisition package must be able to be left unattended, operate reliably, accessed remotely, and be suitably protected from the harsh environment. The system must provide for real-time data collecting and processing to be of value in shaft-sinking operations. Instrument wiring requirements must be minimal to control cost and installation time.

A computerized data collection system for a deep mine shaft is illustrated in Figure 5. The shaft sensors are connected to a nearby (<75 m) NEMA IV computer that stores the data and transmits it to a remote station. This computer may be located underground or at the shaft collar and serves as an intermediate data control and storage point as well as providing preliminary data processing capability.

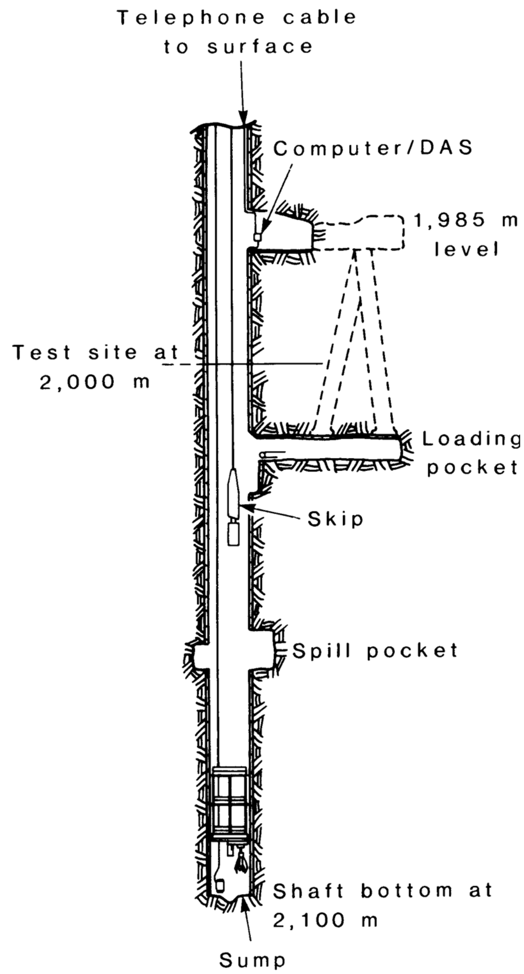

Figure 5. Configuration of data acquisition system for deep shaft instrumentation.

PRELIMINARY ANALYSIS OF FIELD DATA

Field data have been collected from deep metal mine shafts in northern Idaho and western South Dakota. MPBX's, PC's, and SG's were installed in the concrete-lined Silver Shaft at the Lucky Friday Mine, which is 2,000 m deep and 6 m in diameter, and MPBX's in the shaft at the Caladay Project, which is 1,700 m deep and 7 by 3.5 m in cross section (Figures 3–5). Both of these mines are in the Coeur d'Alene Mining District in northern Idaho. Single-position borehole extensometers (SPBX's) and load cells in timber blocking and rock bolts were installed at the 1,150-m depth at the Homestake Mine in Lead, South Dakota, in a 6 by 4 m rectangular shaft (Figure 6). Typical results are presented to illustrate the initial data processing scheme prior to detailed analysis and numerical modeling.

Initial data processing provides a time or face advance plot of the measured parameters over a specified time or distance interval. The plots can then be interpreted to produce a variety of information, including:

- Initial and long-term stability based on rate of change (velocity) or change in the rate of change (acceleration) of the measured parameter with time
- Estimation of elastic/plastic Interface in the rock mass
- Required face advance before the rock mass or support system reaches equilibrium
- Relative effect of geologic structure
- Elastic safety factors of the concrete liner, shaft set, or rock bolt
- Rock displacement/support load Interaction
- Effects of extraneous excavations such as loading pockets, shaft stations, or nearby mining
- Estimate of in situ stress field.

Figure 7A is a time/displacement plot from a test section 800 m deep in the Silver Shaft. It shows shaft wall displacements for MPBX's oriented normal (E3) and parallel (E1) to the steeply dipping bedding planes in the Coeur d'Alene District. There

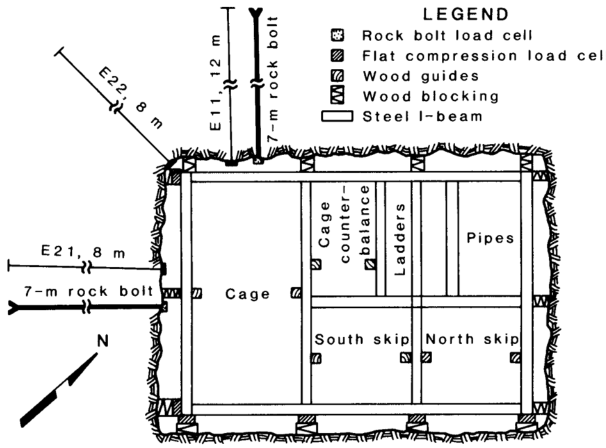

Figure 6. Installation positions for SPBX's and FCLC's in steel sets in rectangular shaft.

was significantly greater displacement normal to bedding than parallel to bedding. Stability is indicated at about 5 days elapsed time and after 19 blasting cycles (about 13 m of advance).

Figure 7B shows a more detailed time displacement plot of the individual anchor points at E3 and indicates an instantaneous and discrete displacement associated with bench blasting on the shaft bottom and a gradual deceleration between blasts. Response characteristics show that maximum displacement occurred at the shaft walls and decreased rapidly with depth. There was also a definite break in magnitude and response sensitivity between 2 and 3 m, suggesting an elastic/ plastic transition zone. Thus measurements of shaft rock mass displacements serve to indicate the effect of geologic discontinuities,

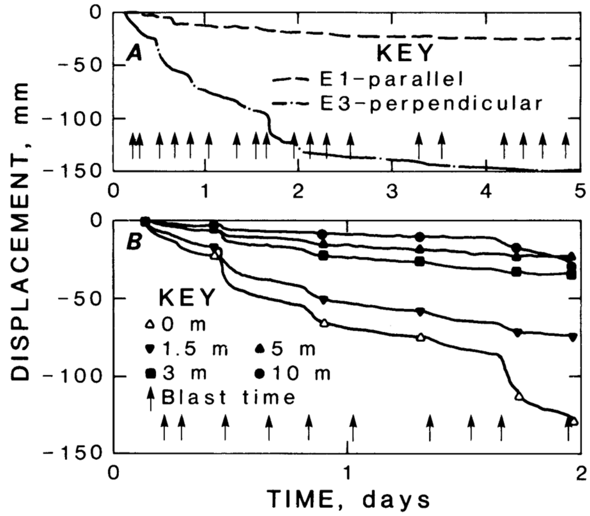

Figure 7. Time displacement plot for (A) shaft wall displacements at 800 m depth and (B) anchor point displacement distributions.

time-to-stability, effect of face advance on stability, and effect of high stress levels, such as development of an elastic/plastic interface.

Figure 8 illustrates several shaft response characteristics for an elapsed time of 30 days. These data are from the 2,000-m level of the Silver Shaft. Figure 8 again shows significantly greater displacement at the shaft wall than at depth. Also an elastic/plastic interface zone is evident between the collar and the 2-m depth. This plot shows that stability was achieved at about 10 days elapsed time. However, at 12 days elapsed time, displacement then accelerated in conjunction with excavation of a loading pocket located about 15 m below the test level.

This effect is particularly visible in Figure 8B, which is a plot of tangential liner stress measured at selected points at the 2, 000-m level (see Figure 3B). The initial 24-hour response shows residual effects of concrete curing, shrinkage stresses, and temperature differential. As the concrete stiffened, substantial pressure began to develop, increased steadily during the next 3 days, and stabilized as the shaft bottom reached almost 13 m beyond the instrumented level. The effect of the loading pocket excavation at day 12 resulted in apparent doubling of liner stress from initially stable levels.

The most striking feature of the liner stress data is the pairing of the load response of PC1 with PC3 and PC2 with PC4. Pressures in the NE-SW quadrants of the liner were considerably greater, suggesting a maximum stress trending NW to SE in the horizontal plane. Initial peak pressure and stability of the shaft liner are indicated as the shaft bottom approached a 13-m distance from the test section. The pressure ratio developed at this point, between the NE/SW and NW/SE pairs, was about 1: 2.5. A simple elastic analysis based on a Kirsch solution shows that a ratio between maximum and minimum applied field stresses of about 1.6 would yield a 2.5 ratio between maximum and minimum liner stresses. This assumes a uniform liner thickness of 30 cm with 60 cm of blast-damaged material surrounding the liner and a reduced modulus about one-tenth that of the intact material.

Strain gauges were placed at corresponding pressure cell locations and at intermediate points (see Figure 3B). The tangential strains measured in the liner at the PC locations are shown in Figure 8C at 30-days elapsed time. As with the pressure cells, initial response during the first 24 hours was due to concrete curing and temperature stabilization. Strain increased steadily through day 6 and leveled off as shaft advance approached 13 m beyond the test section at day 10. The effect of a loading pocket excavation and continued shaft sinking was clearly seen on all strain gauges, particularly on SG's 3 and 4, which lie on either side of the pocket/ shaft intersection.

In addition to verifying the pressure cell response and providing an estimate of concrete modulus, the strain gauge array can be used to estimate principal axes of tangential strain and thus secondary principal stress directions. By applying a least-squares technique to the strain gauge data at selected times, an elliptical fit is obtained. Figures 9A-D illustrate a fit of the data at elapsed times of 10, 20, 30, and 70 days, respectively, corresponding to significant events in shaft construction history. Points 1 through 8 are the tangential strains measured at the strain gauge locations and plotted as vector quantities, with the origin at shaft center. The equation of each ellipse indicates major and minor strain as major and minor (a and b) axes of the ellipse. Figure 9A shows a maximum of 532 microstrain and a minimum of 407 microstrain, with the minor strain axis oriented N 48° W, indicating principal field stresses congruent with this orientation.

During the following 10-day period, the loading pocket was excavated. Figure 9B shows the resultant ellipse generated at day 20. The minor axis has rotated to N 17° W and shows the superimposed effect of the changed geometry. Figure 9C illustrates that the orientation of the stress axes was maintained as shaft sinking commenced. The length of the axes, and thus

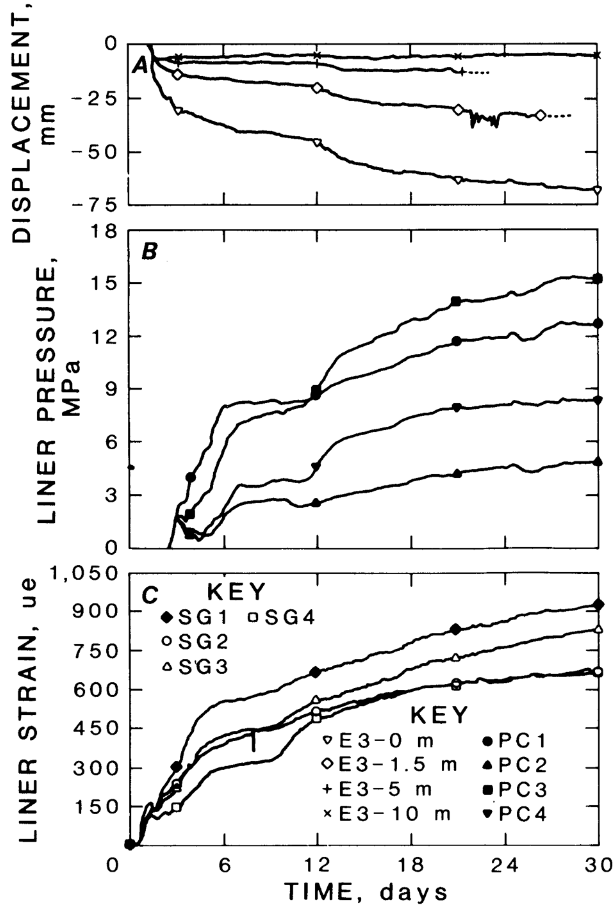

Figure 8. Rock and support response at 30 days. (A) Rock mass displacements at four points in the rock mass, (B) tangential stress at four positions in the concrete liner, and (C) tangential strain at four positions in the concrete liner.

magnitude of major and minor stresses, also increased significantly from day 20 to day 30. Orientation of the ellipse remained essentially the same although magnitude increased through completion of the shaft at an elapsed time of 70 days.

Results of measurement of normal loads in timber blocking of steel sets and axial loads on rock bolts in the Ross Shaft at the Homestake Mine is shown in Figure 10. The load cells were located on adjacent walls of the shaft near the 1,100-m depth to measure shaft set response as shaft pillar mining commenced nearby. The normal loads on the blocking have equilibrated to less than 5% of the crushing strength of the wood blocking. Little evidence of point loading or general shaft closure is evident over the initial 6 months sampling period. It is interesting to note that regardless of initial "blocking-in" loads, all blocking points tended to attain equal load in time, probably due to skip cycling. As mining approaches the test section, more significant shaft set response will be indicated.

APPLICATIONS TO SHAFT DESIGN

There are many factors affecting the structural stability, and thus the design, of a deep mine shaft. "Fixed" conditions include the magnitude, direction, and ratio of in situ stresses, geologic environment, and rock mass properties. Of crucial importance to design is a preliminary evaluation of the in situ stress field and physical properties of the rock mass. "Variable" factors include the size, shape, and orientation of the shaft, the type and dimension of support, and the time lag before support installation.

Specific design questions that need to be answered are (1) what is the minimum liner thickness that can be used and (2) how far behind the face can the liner be carried while a reasonable factor of safety against liner failure is maintained. The liner thickness or installation delay above the face can then be weighed against increased liner cost and the possibility of prelining ground instability.

A design approach which integrates field measurements and numerical models provides realistic design specifications. The procedure is to validate the model by comparison with field measurements. Analysis of shaft displacements are difficult due to the obvious effect of geologic discontinuities. To account for this nonelastic response, a "ubiquitous joint" model has been

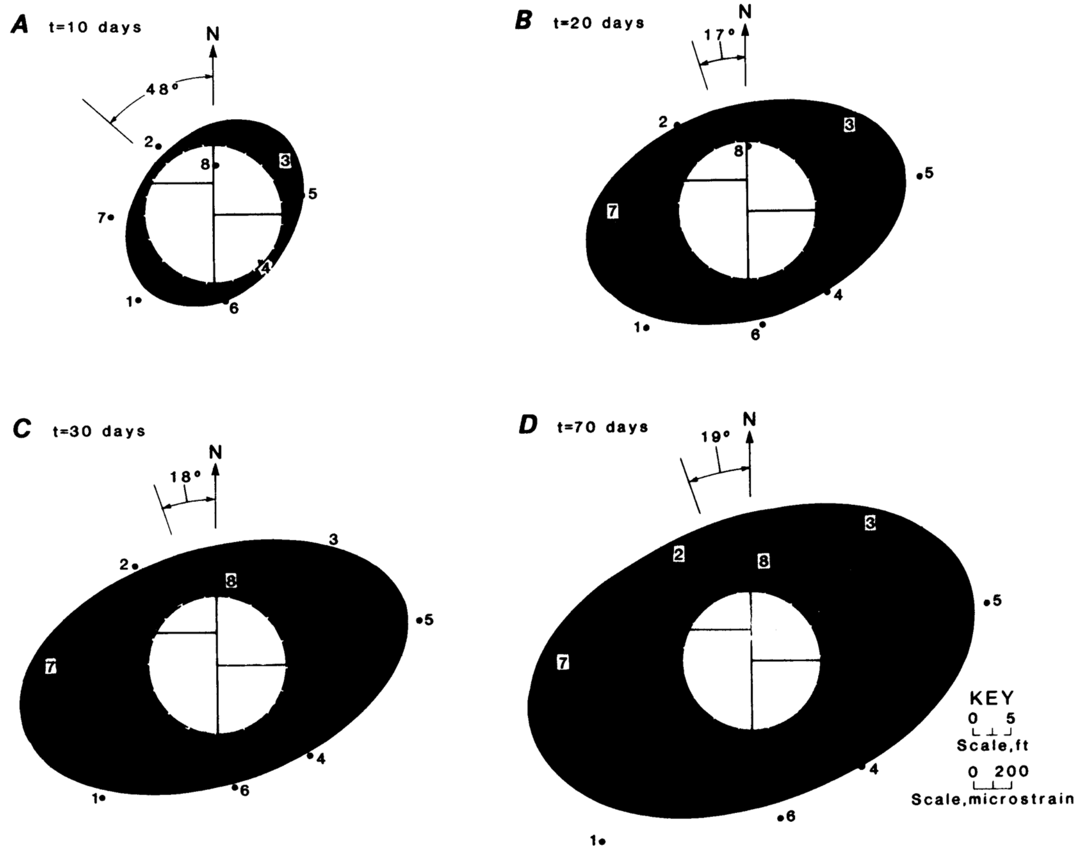

Figure 9. Tangential strain measured in line as a function of elapsed time at (A) 10 days, (B) 30 days, and (C) 300 days.

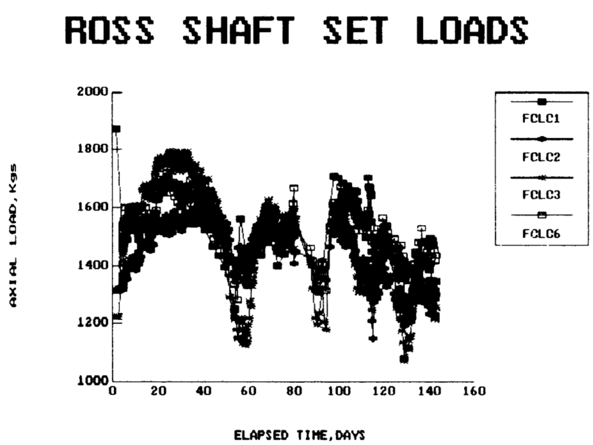

Figure 10. Axial loads in timber blocking in steel shaft sets.

incorporated Into an explicit finite-difference program.[4] In this model, the rock mass is assumed to be a continuum whose material response is governed by a series of oriented "weak" planes within the rock mass.

The rock mass is numerically characterized by joint orientation and material properties. The results are illustrated in Figure 11A where the predicted and actual displacements at the 2,000-m depth of the Silver Shaft are plotted as a function of radial distance. A reduction factor has been applied to the computed data to account for the three-dimensional effects of face advance.[5] This model reasonably reproduces the important feature of large displacement contrast between the direction normal and parallel to structure. Thus, rock mass displacements may be predicted at any location or depth within the shaft with a reasonable degree of confidence.

A primary goal of liner stress analysis is to provide an engineering estimate of the stresses as a function of the in situ stress state. A solution for tangential liner stress[6] consists of hydrostatic (P°) and deviatoric (S°) stress components:

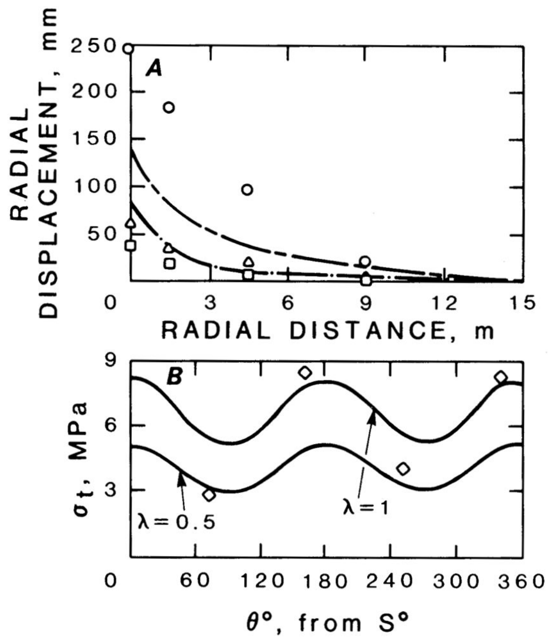

Figure 11. Comparison of measured versus calculated values. (A) Radial displacements versus distance Into the shaft wall and (B) tangential stress versus angle from deviatoric stress direction.

$\sigma_\dagger$= $P°\ S_h(r)+S°\ S_d(r)\cos 2\theta$, where $S_h(r)$, $S_d(r)$=Assumed functions describing stress conditions at the rock-liner Interface,

and θ = angle from deviatoric (S°) stress direction.

Two parameters have the most important effect on the tangential stress: the concrete/rock mass shear moduli ratios (λ) and the ratio of liner thickness t to outer shaft radius a. In the actual field case, the thickness of the liner varies with angular position about the shaft due to overbreak. Figure 11B shows the tangential stress plotted as a function of angle from the deviatoric stress direction for a concrete thickness of 90 cm. Actual field measurements from the 2,000-m level of the Silver Shaft indicate the same maximum and minimum locations as predicted by equation 1; the maximum stress occurs at ±90° from the principal horizontal stress direction. Also, the field data are bounded by the appropriate moduli contrasts between concrete and the rock mass.

The above model accounts for the general variation of the tangential stress and can be shown to bound the field data for the field moduli ranges estimated. Thus, an appropriate mechanism for evaluating liner thickness and support delay for depths and rock mass properties may be developed. A plot illustrating the relation of t/a to the normalized average tangential stress at its maximum position $(\sigma_\dagger\ (max)\phi P°)$ for various values of the shear modulus ratio is given in Figure 2A. For a given hydrostatic stress component P°, support delay factor ϕ and shear modulus ratio λ, the required liner thickness can be calculated. The value of $\sigma_\dagger$(max) can be calculated and compared to the concrete i iner strength (assuming that no tensile stress exists) with any desirable factor of safety.

Plots were developed for the specific case of the Silver Shaft that illustrate the required support delay in terms of shaft radius as a function of depth for various ratios of t/a and λ as shown in Figure 12B. These figures assume a concrete compressive strength of 28 MPa and a factor of safety of 2.0. The bounding value of λ=1.0 shows that for a support delay of one diameter, the maximum depth before liner failure is about 2,700 m.

The specific field conditions encountered in sinking an actual shaft will vary from the ideal ized case presented here in several respects. First, the liner is generally placed at least one shaft diameter behind the face. At this point, most of the elastic rock mass displacement has already occurred. For a poured concrete liner, the concrete also remains “green” for some time as face advance continues. At an advance rate of 2 to 3 m/d, the face can advance two diameters in less than 1 week, and thus displacement equilibrium can occur before maximum concrete strength is reached. This effect is offset to a certain extent by the use of additives that promote high early strength in concrete.

Second, the radial stresses exerted on the liner can be highly nonuniform as a result of geologic conditions and structural control of rock mass displacements or a nonuniform stress field. Also, there is a practical limit of how far behind the face the liner can be poured; this limit varies from operation to operation. Loose material must be adequately supported at the face to eliminate the hazards of falling ground. Depending upon conditions, mesh and bolts used for preliminary support may be adequate. Attempts are being made to simulate these construction variables in more sophisticated numerical models.

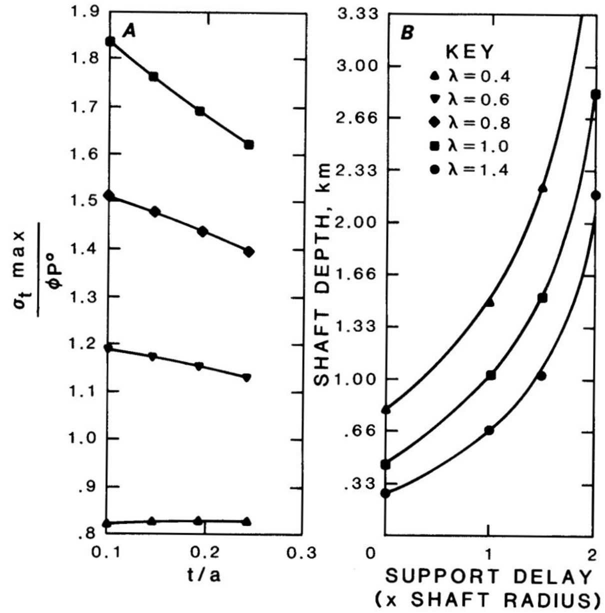

Figure 12. Plots of various shear modulus ratios of concrete to rock mass, (A) normalized liner thickness (t/a) to normalized tangential stress $(\sigma_t/\phi P^{o})$; and (B) support delay for various shafdepths for t/a=0.1.

SUMMARY

Measurements taken during construction of a deep shaft provide many important findings. These include bedding planes and a high, unequal stress controlled displacement. Most displacement occurs within 2 m of the shaft walls and decreases rapidly with radial distance. Initial displacement stability is achieved as the shaft bottom reaches 13 m below the test level. Secondary excavation, such as loading pockets, stations, etc., can be expected to influence rock movement and liner pressure at least as much as does shaft sinking.

Liner stress and strain suggest trends of major field stresses. Initial liner stability is achieved when the shaft bottom is about 13 m distant, or two shaft diameters, correlating with displacement stability and elastic rock response. Secondary excavation causes liner stresses and strains to increase and reorient. Continued shaft sinking causes redistribution of stress and strain response in the liner. A statistical analysis of liner strains shows that the major and minor stresses acting on the liner are biaxial. These data also are reliable indicators of liner response, particularly as secondary excavation disturbs the initial rock and liner stress equilibrium.

Relatively simple numerical models can be used to bracket measured response and provide shaft design criteria. A ubiquitous joint model accounts for variation in rock mass displacements due to anisotropy and predicts design displacements. An elastic, no-slip, rock-support interaction approach can be used to bracket measured liner response and develop liner thickness and support delay guidelines. Based on a combination of observation, analysis, and empirical approaches, overall behavior of the rock mass around a deep mine shaft can be determined.

References

1. McKinstry, B.A. Sinking the Silver Shaft. In: Proceedings of the 1983 Rapid Excavation and Tunneling Conference, vol. 1 (Chicago, illinois, June 12–16, 1983), pp. 493–512.
2. Beus, M.J., and Board, M.P. Field Measurement of Rock Displacement and Support Pressure at the 5,955-Ft Level During Sinking of a Deep Circular Shaft in Northern Idaho. Bureau of Mines Report of Investigations 8909, 1984, 11 pp.
3. Corso, W.T. Mine Planning of the Ross Shaft Pillar, Homestake Mine, Lead, S.D. In: International Symposium on Mine Planning and Equipment Selection, Calgary, Alberta, Canada, November 3– 4, 1988.
4. Cundall, P.A. User's Manual and Report on NESSI, Version 1.145. Shell Research KSEPL, Rijswijk, Netherlands, 1983, 70 pp.
5. Schwartz, C.W., and Einstein, H.H. Inspired Designs for Tunnel Supports: Vol. 1—Simplified Analysis for Ground-Structure Interaction in Tunneling. U.S. Dept. of Transportation, Rep. DOT-TSC-UMT7–80–27. I, 1980, 450 pp.
6. Detournay, E. Two-Dimensional Elasto-Plastic Analysis of a Deep Cylindrical Tunnel Under Non-hydrostatic Loading. Ph. D. thesis, University of Minnesota, Minneapol is, Minnesota, 1983, 133 pp.

Extensometer investigations of frozen shaft lining

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SYNOPSIS

After 25 years of investigations of deep shafts which have been sunk using freezing techniques in heterogenous weathered and saturated rock masses of Legnica-Glogow Copper Basin (LGOM), it is possible to characterise the load changes in the shaft lining at all stages of sinking. Observations from damaged linings in shafts sunk with freezing are presented as well as data obtained from strain gauges and other instruments installed in the lining and the frozen rock mass around the P-6 shaft at Polkowice Copper Mine.

The pressure on the lining exceeds that which would have existed if the shaft was sunk using traditional methods. The high pressures are attributed to a volume increase during freezing of the rock mass and structural damage to the rock mass after thawing.

INTRODUCTION

The superficial cover in the Legnica-Glogow Copper Basin consists of fractured and weathered rocks, which are unable to withstand the stress concentrations during shaft sinking. This low strength requires that the rock is consolidated and strengthened prior to sinking. This is achieved by freezing.

During the sinking process, there are several stages of freezing which result in variable loads on the shaft lining. The mechanical properties of the fractured and highly waterlogged rock mass change radically during the freezing process.

Freezing also results in a change of the rock mass equilibrium due to the volume increase during each stage of shaft sinking. Thawing results in further changes to the loading on the lining.

The final state of stress in the rock mass is a result of complete thaw and results in stable conditions unless equilibrium is disturbed by further excavation.

The behaviour of the rock mass is extremely difficult to anticipate during the freezing and thawing processes especially when the material consists of clay, loam or mudstone. It was observed that there were varying degrees of damage to shaft linings due to the freezing/thawing at different locations. This damage could be reduced or avoided if the mechanisms causing the damage were better understood, loading of the lining could then be reduced by controlling the freezing and sinking operations.

This paper presents some of the extensive research and observations that have been made of this phenomena of variable loading of the rock mass and shaft lining in the Legnica-Glogow CopperBasin.

GEOLOGICAL CONDITION

The freezing process is applied to rocks of the Cenozoic and upper Triassic sandstones locally known as the Bunter Sandstones.

The Cenozoic complex consists of alternating sand and clay layers of varying consistency. In addition there are several lignite and lignite dust mixtures interlayered within the clay and sands of the Tertiary formations.

The rate of freezing is highest in sandstones and lowest in lignite for the above rocks. The clays show highly variable freezing rates and frozen strengths. Clays with lignite dust mixtures have a very low freezing speed. The laminated clays have the lowest frozen density with a strength of 0.8 to 2.0 MPa at a temperature of −20°C. This low strength is attributed to unfrozen water remaining within the clay and which facilitates inter-molecular slips leading to shaft deformations.

Results of test and investigations indicate that rocks with a low angle of internal friction are most inclined to exhibit creep properties. With freezing it has been found that the internal friction angle decreases for some clays at LGOM with values of less than 1.5°. The freezing affect for sands, on the other hand, increase the strength to up to 20 MPa.

Freezing complications at LGOM are caused by:

– the presence of alternating plastic clays and quicksand with between 17 and 40 saturated areas discharging approximately 2300 m^3 per hour of water.

– the presence of slow freezing interbedded lignite occupying 20 to 50 m of the 600 m frozen zone in the 1200 m deep shaft.

– the presence of fractured and highly waterlogged sandstones at depth. The fast freezing of the sandstones causes the low thermal conductivity layers to slow down the advance of the freezing front.

THE INFLUENCE OF THE FROZEN MANTLE ON SHAFT LINING

The frozen mantle is formed around the shaft to protect the excavation from water or quicksand inrush as well as to provide the proper conditions for assembling the lining. The mantle also serves to maintain the integrity of the lining during freezing and thawing. The mantle should have sufficient strength, and extend far enough to cover the zone ahead of the sinking shaft. If these conditions are not met then failure of the lining could occur at unfrozen sidewalls.

If the frozen mantle continues to develop after the lining is installed the rock column will increase and additional loading of the lining will occur. This overloading will be in excess of that which would exist due to depth and the mechanical properties of the rock mass. The overloading may result in fracturing of the concrete lining and buckling of the outer tubbing concrete layer, as well as the unsealing and squeezing of lead gaskets at the tubbing joints.

Furthermore the frozen mantle functions are not terminated with completion of sinking. The mantle continues to be used as a necessary and efficient protection during scaling and eventual repairing of the lining.

It is relatively simple to rehabilitate the outer tubbing concrete when the frozen mantle still exists even if the concrete is quite buckled. The renovation of the concrete lining, however, is more difficult (Fig 1).

The lower freezing speed of the lignite results in temperatures above 0°C when the temperature of other rocks is far lower (Figure 2). Lignite, even when not sufficiently frozen, is far stronger than clays. Nevertheless, the deformation of the lining as indicated by unsealing of tubing joints, squeezing of lead gaskets or fracturing of the concrete, is observed in the zone of the lignite strata.

The extent of the frozen mantle of the R-5 shaft, which was reached after 5 and 10 months of freezing, is shown in Figure 3. After 5 months the degree of freezing is adequate for 200 m of shaft sinking but will not be adequate for lining assembly. The point 2 on this figure indicates the depth that is suitable for lining installation. Below 170 m it may be impossible to achieve a good quality lining due to insufficiently frozen ground. If the rock mass is inadequately frozen the concrete is often contaminated with rock spalling from the sidewalls. It is also difficult to maintain dimension control due to the large deformations.

The mechanism of additional loading to the lining in freeze sunk shafts has been estimated from many observations. A short description of the process is given:

– hydration of the poured concrete is exothermic producing up to 21 MJ/m^3 of concrete. This leads to defrosting of the mantle adjacent to the concrete mass.

– the frozen mantle loads the defrosted part of the mantle. This leads to consolidation of the soil and reloading of the lining. Fractures and microcracks superimposed with contraction fractures develop.

– refreezing of the defrosted mantle then reloads the lining further.

All of these phenomena are intensified when the shaft sidewalls have not been adequately frozen before the lining is installed. This may result in cracking or even buckling of the concrete lining. Localized buckling of the concrete lining has been observed on the P-2 and R-1 shafts and been measured using instruments in the P-6 shaft. These factors indicate that the tubbing lining is indispensable at LGOM with the state of present day technology and the existing ground conditions.

THE CHARACTERISTIC OF SHAFT LININGS AT LGOM

Several combinations of concrete lining methods and freezing and sinking technology have been tested at LGOM. The lining types may be differentiated by their construction and method of installation. These are listed in Figure 4 along with the web value and the final lining pitch.

Currently there are two basic types of lining for 7.5 m diameter shafts used in the frozen zone at LGOM: brick-concrete lining of C type (down to 50 m depth) and tubbing concrete lining of F type which is used below 50m depth. Linings of A, B an D type are not used due to inappropriate installation technology for LGOM conditions. Lining of E type was found to be too labour consuming due to its large outbreak requirements.

REASONS FOR LINING DAMAGE IN FROZEN GROUND

From a literature survey, observations of 27 shaft lining and a detailed investigation of P-6 shaft, it is possible to list 9 main reasons for damage to linings:

– excessive shrinkage of concrete due to unsuitable quality and quality of components

– freezing of lining before the cement has set. This has been observed in thin layers of B type lining that include a steel plate insert. After thawing the lining does not reach its proper strength

– structural changes in the concrete caused by freezing of excess water in saturated concrete

– premature loading of lining before the concrete has built up sufficient strength. This causes microfractures and fractures.

– local overloading of lining adjacent to plastic rock layers. Creep of the rock occurs particularly during heat input from the concrete hydration.

– shut down in freezing of the mantle which may result in excessive damage to the lining or even loss of the shaft

– non-uniform loading which may be due to an increase in volume of the frozen mantle. This occurs when the frozen zone does not include the zone of current excavation or lining installation. These loads are also a result of refreezing of mantle that was previously defrosted by the heat from current hydration.

– point load due to high pressures during cementation.

– sudden increases in water pressure caused by collapse of frozen mantle during thawing (this was indicated by wire extensometer measurements in the P-6 shaft lining).

ADDITIONAL IMPORTANT RESULTS OF DEFORMATION INVESTIGATION P-6 SHAFT LOCATION OF MEASUREMENT POINTS AND RECORDED RESULTS L.C.

In the P-6 shaft of Polowice Copper Mine, 181 strain gauges at 6 main and at a few auxiliary measurement levels were installed in different rock strata (Figure 5).

The frozen zone depth of the shaft has reached 425m. Sensors were fixed during the sinking process. All rock layers have been frozen except for those including lignite. An example of the set of sensors along with a cross section of lining construction at a main measurement level is shown in Figure 6. Measurement were usually made twice per week but during particular stages such as cementation they were read as frequently as once per day,

Six parameters were read by the sensors, these were:

1. strain in three directions inside the brick and concrete lining (vertical, radial and tangential)
2. strain in two directions at tubbing surface (vertical and horizontal)
3. pore water pressure within the rock mass and concrete
4. vertical deflection of shaft lining in two orthogonal planes
5. Opening of wooden ring gaskets between tubbing columns (measured at 3 points on the shaft circumference)
6. Temperature inside the concrete and at the lining-rock contact surface

Concurrent observations were made of the entire shaft lining as well as the shape and range of the frozen mantle with temperatures of the rock. The entire measurement scheme took 9 years while additional investigations of more than 20 shafts have taken 25 years. Similar stages of the development of lining load could be estimated for each shaft.

As an example the P-6 case is presented, where the stages of lining load are:

Stage I	shaft sinking within the frozen rock mass
Stage II	shaft operational when freezing is still ongoing (this period was unusually long in the P-6 case due to exceptional demands on the Copper Mine)
Stage III	thawing of rock mass and cementation period
Stage IV	shaft operational period when the lining is fully loaded by water pressure and the defrosted rock mass.

Maximum strains in the lining take place during Stages I and III

TIMING OF LOADING CHANGE IN LINING

It was concluded from the measurements that there are 3 main stages when an increase in loading of the frozen shaft lining occurs:

- when the cement hydration process is finished the defrosted zone is refrozen and the lining load increases due to increase in frozen rock volume
- during cementation point loading may occur when the large quantities of slurries are forced in at pressures of up to 7MPa
- after shut down in freezing of the mantle when the entire hydrostatic pressure is affecting the lining

These events result in large changes in the strains recorded in the lining as indicated in Figure 7. Stresses calculated from the strain changes are recorded.

The sudden water pressure increases as a result of shut down of freezing of the mantle and is indicated in Figure 8. The shaft lining was not loaded by water prior to shut down. Corresponding to this freezing shut down is an increase of water pressure within the concrete lining. This pressure is exerted directly on the tubbing lining.

The water pressure inside the rock mass and concrete is constant and approximately equal when the tubbing lining is tight. The oscillating shape of some parts of the concrete pressure diagrams (Figure 8b 365 and 374 m depth) is a result of leakage from the tubbing lining and waste material within the tubbing lining.

Similar behaviour was proposed for the defrosted layer of lignite at 374 m depth (Figure 8a). The material was originally non-water bearing until the fractures associated with freezing and thawing opened sufficiently for periodical water migration. The time period for each loading stage (Figure 7) is different for particular shafts. The periods will depend on the method of ground freezing, the sinking technology and the rate of thaw. The thawing process does not occur simultaneously along the entire length of the mantle but rather proceeds from the bottom to the top. This can result in it taking a year for the passage of water pressure loading from the bottom part of the lining to the top of the shaft column.

ROCK MASS PRESSURE ON THE SHAFT LINING

Formulas for rock pressure calculated on shaft linings are grouped according to the hydrogeological conditions. There are however, no univertall applicable formulas.

Satisfactory results can be obtained using calculations in accordance with the Branch Standard BN-79/0434–02 entitled “Shaft Lining, Loadings”. The calculations process is very labour intensive because of the wide range of parameters such as internal angle of friction, cohesion, porosity, etc. Some generally accepted coefficients for rock pressure as a function of depth are:

- saturated sand 0.013H, MPa
- clays 0.017H, MPa

where H is the depth in meters

The load factors are used in conjunction with a constant for shafts with a larger than 7.0 m excavated diameter. For example for a 9.2 m excavated diameter the general coefficient is 0.014 H for sand and 0.0185 H for clays.

Results of insitu measurements by Hilby (1971) in West Germany indicate that rock pressure on the shaft lining reached the value of 0.0194 H.

Investigations made with instruments within clays at LGOM have given a coefficient of 0.02045 H. It appears that insitu measurements and calculations from the Branch Standard are compatible if the correct internal angle of friction is utilized. The rock pressures on shaft linings for the rocks at LGOM have been calculated as it shows in Figure 9. Maximum values of pressure calculated according to the Branch Standard are:

- for H=250 m depth (Poznanskie clays, Pliocene) p=5.08 MPa
- for H=450 m depth (overcoal clays, Miocene) p=9.15 MPa
- for H=815 m depth (sandstones, Bunter Sandstone) p=10.5 MPa

The calculation above shows that the largest lining pressure will exist in the overcoal clays. Within these clays the pressure could be given as different values depending upon which method was used:

- p=0.0185H=8.32 MPa in accordance with former settlements
- p=0.02045H=9.2 MPa in accordance with the insitu measurements
- p=0.02033H=9.15 MPa in accordance with calculations based on the Branch Standard

From measurements of lining strains the following changes in rock pressure were recorded:

- Stage I (freezing) 0.00987H, MPa
- Stage II (sinking) 0.01281H, MPa
- Stage III, IV (thawing and cementation) 0.02045H, MPa

At stage IV 63% of the lining load is due to rock pressure which amounts to 0.01295H and the rest (37%) is caused by water pressure which amounts to 0.00750H. The values quoted above are not identical for each shaft but do illustrate the process of load variability with each stage.

It should be pointed out that the rock pressure effect on shaft linings is still not always calculated correctly. It is accepted that the greatest pressure on the lining is due to the free water within sands, gravels or sandstones. The pressure from clays is often neglected yet it can be greater than that from the saturated sands.

Despite the fact that the pressure from the clays is calculated as neglible the shaft linings are sufficient strong to accept the greater loading. One can conclude that a well constructed concrete-tubbing lining is able to endure pressures in excess of its design capability. The measurements from the strain gauges indicate that the safety factors in the tubbing lining reach a value of 10 when the buckling stress is 1000 MPa while the stresses in the concrete exceeded the maximum allowable values.

It is evident that current linings could withstand even higher stresses if the strength and tightness of tubbing joints could be improved. A further conclusion based on direct observation and deformation measurements is that the rock pressure within clays and sands or mudstone that is heavily contaminated with clay is much larger than in saturated sand and quicksand layers. This fact should be taken into account when the sinking technology is decided upon.

TIGHTNESS OF LINING

The lining should be tightened following the sinking operations. The integrity of the lining is often lost (as was described in discussing the pressure affect) and it will require consolidation and packing after defrosting prior to the shut down of the frozen mantle. The mantle continues to function as an effective and indispensable protection during packing and renovation of the lining. If the lining is not packed sufficiently (especially in saturated rocks) then it may not act as a monolithic structure capable of withstanding the full rock pressure after thawing.

The packing and cementation operations should be carried out within the lining and in the rock mass in the immediate vicinity of the lining. The ideal situation is when the defrosted zone extends 0.5 m to 1.0 m behind the lining with the rest of the mantel completely solid.

When the lining is drilled to the interface the grout should be forced in with a pressure no greater than 1.3 times the hydrostatic pressure value. It is important to choose the correct injection pressure. If the pressure is too low then the lining will not be tight enough, while if it is too high a pressure could cause extension of micofractures within the lining. Damage to tubbing segments are possible in the event of a pressure surge. Such damage at LGOM has been described in publications.

It is difficult to compare the cementation operations in the various shafts at LGOM. The initial cementation operations did not always follow standard procedures due to technical or organisation restraints. The quantity of cement grout emplaced has varied between 0.65 and 1.25 tons per m of shaft with no dependence on shaft diameter.

After the major cementation phase it is often necessary to recement in zones of high water discharge. Additional repacking is required in downcast shafts where the linings are periodically frozen and thawed due to seasonal temperature changes.

All the shafts at LGOM appear sufficiently tight after defrosting following the full treatment of cementation and possible renovation. The average water inflow varies between 3 and 10 litres per min per 100 m of shaft length.

SUMMARY

There is a correlation between the loading of the shaft lining and the extent of freezing of the rock mass during freeze sinking of the shaft.

Thus a regular, strong frozen mantle should be formed within which, the line of the shaft or even the whole inner surface of the excavation (within weak rocks) should be closed.

The integrity of the concrete is usually lost in frozen shafts. This requires that cementation and renovation work is carried out after the initial thaw but before complete defrosting of the mantle.

An adequately tight lining after thawing is a measure of a high standard of work. The usually rapid increase in water pressure associated with the shut down of the frozen mantle will highlight even the smallest defects in the lining.

Investigations of concrete setting temperatures within different lining types is evidence that the concrete in LGOM shafts is not freezing prior to it setting. In a shaft with a primary brick lining where concrete was placed in a single pour to a height of 30 m, the temperature in the concrete reached 76°C. When placing the concrete in rings of 1,5 m and 3,0 m height the

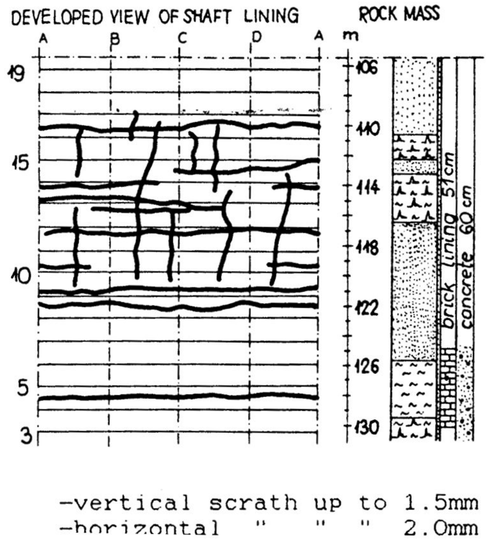


Fig.1 Cracking damages of concrete lining of P-6 shaft.

concrete maintained a positive temperature for 2 or 3 weeks despite highly frozen sidewalls. In downcast shafts which are subject to periodical freezing it is necessary to heat the air in winter.

Early recognition of the existing conditions and proper treatment has enabled the shafts at LGOM to have good tight linings that fulfil their specifications.

RERERENCES

1. P.Hilbig, H.Kratzch, H.Hofman. Diemesstechnische Uberwachung eines absatzweise gefrorenen Schachtes. Opladen: Verlag, 1971
2. A.Mitzel, J.Wlodarczyk. Analiza stanu naprezen i wytezenia obudowy szybu, opracowana na podstawie badan nieniszczacych. Wroclaw: 1975.
3. I.Plesniak, B.Przygodzka. Water pressure during the defrosting of shafts sunk by special methods. Underground Construction. no.3, 1974.
4. I.Plesniak, B.Przygodzka. Some Aspects of Loading the Casing in Frozen Shafts. Budownictwo Gorniczo-Przemyslowe i Kopalnictwo Rud. no.4, Czestochowa, 1978.
5. I.Plesniak. Rozwoj plaszcza mrozeniowego w swietle praktycznych doswiadczen glebienia szybow LGOM. Doctor's thesis. Krakow, 1978.
6. B.Przygodzka. Wybor najkorzystniejszej metody rozmrazania szybow na przykladzie praktycznych doswiadczen LGOM. Doctor's thesis. Krakow, 1978.
7. Zaklad Budowy Kopaln—KGHM Lubin. Collection of working plans of measurements and investigations of frozen shaft linings, freezing and thawing processes at LGOM. Lubin, since 1964 till 1985.

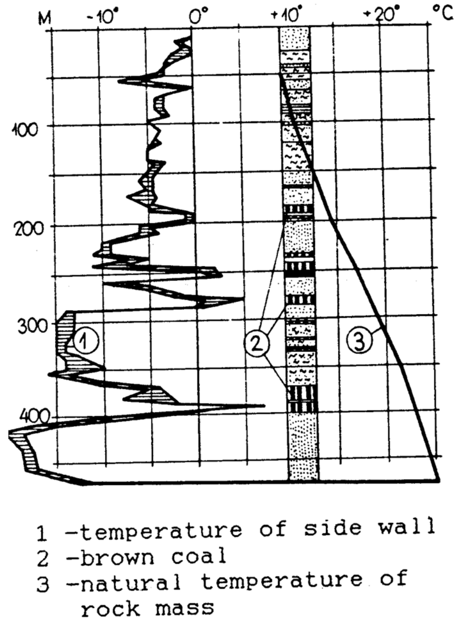


Fig.2 The range of side walls temperature of sunk R-2 shaft with plus temperature in lignite groups.

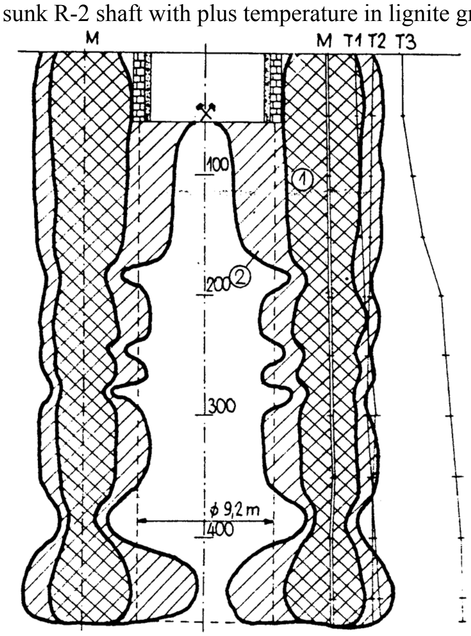


Fig.3 Frozen mantle of R-5 shaft after:

1–5 months long freezing

2–10 months long freezing

Assem-bling way	Shaft web walue / final lining pitch	Break-out area	Shaft dimen-sion	LINING TYPE		
	3,0 i 15÷25 / 3,0 i 15÷25	40,7	6,0	monolithic concrete	0,60m	A
from down to up	1,5 / 15÷25	47,8	6,0	concrete with steel insert	0,25÷35 0,60m	B
from down to up	1,5÷3,0 / 20÷30	55,4 76,9	6,0 7,5	concrete and preliminary brickwall	0,10 0,51 0,60m	C
from down to up	10÷35 / 10÷35	43,5	6,0	tubbing and concrete	0,60 0,20m	D
from down to up	1,5÷3,0 / 20÷35	55,4	6,0	tubbing, concrete and brickwall	0,10 0,51 0,10 0,20m	E
Underslung tubbing	1,5 / 1,5	66,4	7,5	tubbing and concrete	0,60 0,25m	E

Fig. 4 Types of shaft linings used in frozen zone at LGOM

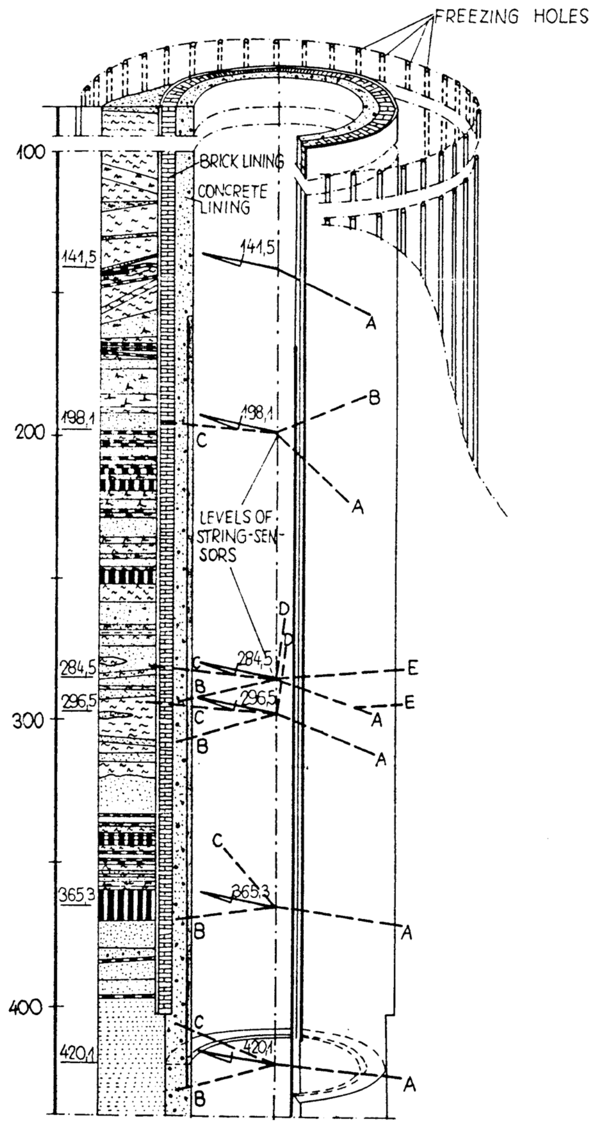


Fig. 5 Localization of the main levels of string-sensors measurements in P-6 shaft

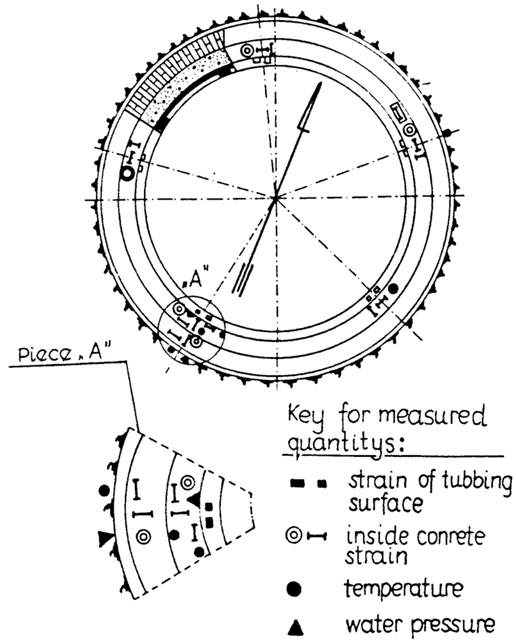


Fig. 6 Location of string-sensors inside the shaft lining at main measurement level

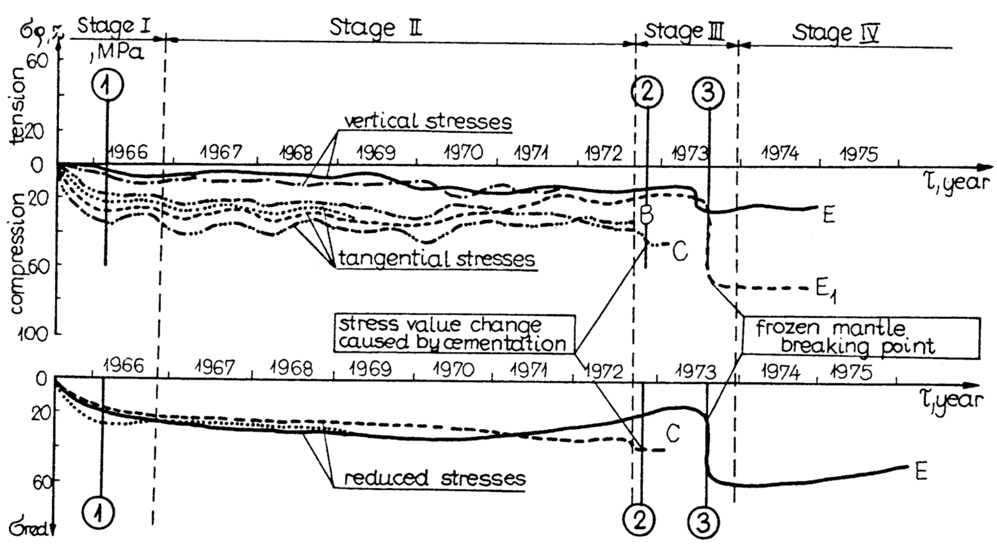
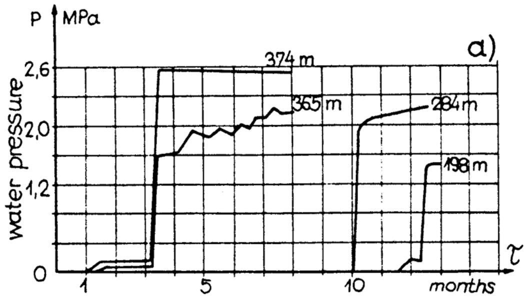


Fig. 7 Stress pattern inside concrete lining of shaft P-6 at the 284,5 m level, as a time function.



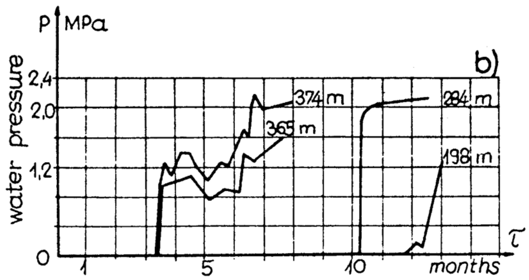


Fig. 8 Water pressure development in the shaft after freezing was shut-down:

a) – in rock mass

b) – inside concrete 1ining

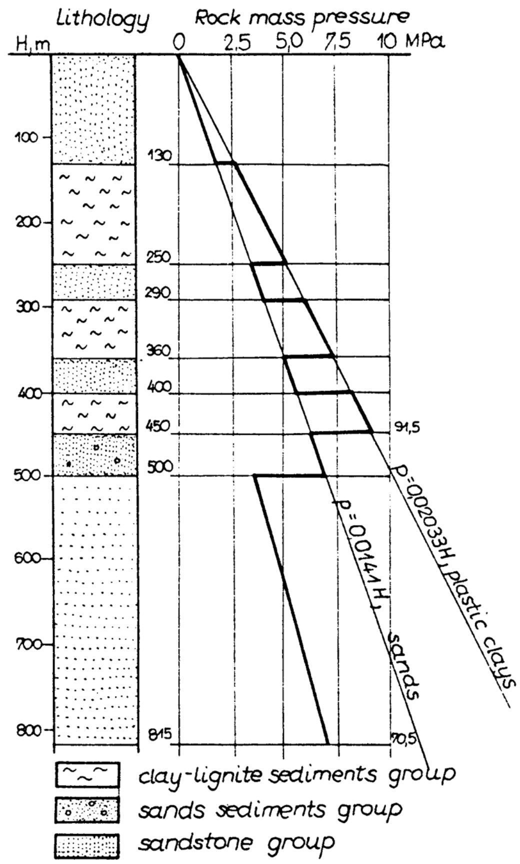


Fig. 9 Diagram of rock pressure at shaft lining in provided for freezing zone at LGOM.

Utilization of a reinforced concrete permanent shaft tower for shaft sinking with the application of ground freezing

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SYNOPSIS

It has been usual practice in Poland for a long time to construct permanent reinforced concrete shaft towers before the execution of the shaft, so that they were used for sinking with the simultaneous carrying out of assemble works in its interior. For the first fime a decission was made to apply ground freezing for shaft sinking below a RC shaft tower. The geological reconnaissance proved the existance of waterbearing levels with water inflow upto 3,78 m/min. Ground freezing was applied to the depth of 260 m.

In order to accomodate the freezing cellar around the shaft collar, a special grid structure of foundations was proposed with bases founded outside the effected zone of freezing. Excavation of foundations was started as soon as freezing holes had been drilled. Final completion of foundation construction was proceeded by sinking down polyethylene pipes and assembly of the brine distribution ring. Having completed the wall slide, an intermediate floor /+30 m/ has been built to separate the lower parts of the tower. Bench marks were installed in the walls to monitor tower subsidence during wall construction and ground freezing. Details on results of monitoring are presented in the paper.

1. Introduction

Shaft construction as well as assembly of headframe and permanent hoisting installation is a very time-consuming operation in the overall investment cycle in the development of a new mine.

In case of execution of a reinforced —concrete shaft tower in a traditional “in series” method of work organization, this cycle lengthens additionally because of the assembly of permanent winding machines only when the tower erection works are over.

It has been usual practice in Poland for a long time to construct reinforced concrete shaft towers before the execution of shaft, so that it could be used for sinking with the simultaneous carry ing out of assembly works in its interior,

The benefits of such a solution are obvious, yet it presents considerable risks to the headframe which is such an important structure. Many unique problems occur especially if the shaft sinking is to be carried out with ground freezing.

Ground uphesvals due to the increase of volume of frozen ground may result in the destruction of the foundation structure. In case of ground freezing up to the surface, the influence of ground upheavals may be considerably reduced or eliminated by means of locating the head-frame’s foundations beyond the frozen ring of ground. However, it must be stated, that such a solution is connected with more foundation works.

The benefits derived from the realization of this undertaking are much bigger in relation to the expenditures incurred due to increased RC works.

The solution utilizing the permanent shaft tower for the needs of shaft sinking with ground freezing method, has been applied in Poland, at shaft No II in the “Czeczott” hard coal mine under construction. It has been a novelty in Polish mine construction. In obtained a certificate of authorship concerning utility pattern as well as patent rights.

2. Characteristics of geological conditions and freezing parameters

The “Czeczott” hard coal mine has been designed as a two-level mine with two production levels.

– Ist level at the depth – 500 m.

– IInd level at the depth – 650 m.

The deposit is to be approched by means of four shafts with an inner diameter of 7,5 m, each.

Three of them have been located in the main part of the mine in the south-western part of the mine ares, whereas the fourth shaft—upcast and service shaft has been located at the distance of about 3,0 km from the main shafts.

Access to production levels will be mined in rocks, by means of drifts and cross-cuts. Longitudinal or transverse advancing longwall system with common use of mechanised shield support has been designed for the exploitation of seams. The target net output of the mine will be 24.000 t/day. On the site of shaft sinking, the presence of the following formations has been determined bused on the exploratory bore-holes drilled earlier:

– Quaternary – from the surface to the depth of 39,0 m
– Tertiary – from the depth of 39,0 to 230 m
– Carboniferous – belowe the depth of 230 m.

Advance bore-holes, drilled in the vicinity of main shafts have proved the existence of difficult hydrogeological conditions, requiring the application of special sinking methods. In the hydrogeological documentation, 13 water-bearing horizons have been distinguished, with varying water inflow, ranging from 0, 174 − 3.781 m^3/min.

In this case, it was decided to drill the first freezing hole as an exploratory -freezing hole, down to the depth of 510 m.

Based on this, hydrogeological conditions have been determined in detail. Analysis of the results from drilling of this bore-hole have proved more favourable mining and geological conditions below a depth of 270 m. However, it has confirmed the difficult and complicated conditions in the overburden layers. Overburden in this region consits of loose rocks, flooded in considerable sections.

Directly under the tower foundation below the depth of 8,0 m the following formations have been found:

- thickened medium grain sands with a thickness of 5,0 m
- thickened gravel with a thickness of 3,0 m
- semi-compact powdery clay with thickness upto 9,5 m.

Basing on the results of exploratory bore-hole, the depth of ground freezing has been determined up to 263 m.

The basic parameters of the freezing process for shaft II are as follows:

- freezing mantle thickness—E=3, 0 m
- diameter of freezing ‘holes circle— D_m=14 m
- number of feezing holes—n=44 pcs
- average t temperature of freezing mantle −258°K /−15° C/
- quantity of heat to be carried away from the ground −2.730.649.617 kcal /1,14×10^{13} J/
- diameter of freezing pipes—D=140 mm
- diameter of down pipes—d=75 mm
- type of freezing unit−8 W-200/2
- number of freezing units −3 pcs
- temperature of brine- −30°C
- time of freezing—4 months.

In June 1978, drilling of freezing holes commenced at shaft II, using drilling rigs, type BU-75. Average drilling rate, obtained for freezing holes, in case of a failure-free operation was about 85 m of hole/day. Connecting of pipes was done by means of welding according to the worked out technology, checked in practise.

The leak-proof tests, conducted in the sunk freezing pipes, have proved the necessity of lowering emergency pipes, dia ∅ 102 mm, in 4 cases.

Additional holes due to excessive hole deviation, have not been drilled. Results of freezing holes plumbing are presented on Fig. 1.

3.
Execution of the reinforced congrete shaft tower

The RC shaft tower, selected for erection had the height of 101 m. Two winding machines have to be installed on the height above 60 m. Each of the winders serves in a separate compartment a skip container of 30 tonnes capacity, Fig. 2. Four RC pylons, which have walls ond floors to a thickness of 30 cm constitute the tower structure. Skips are hoisted in a steel truss, rigidly connected to the RC structure of the tower and shaft.

Total weight of shaft tower and installations mounted inside amounts to ca. 17.000 tonnes. Emergency load i.e. the breaking load for the hoisting rope amounts to 720 tonnes, which constitues 4, 2% of dead load only.

To eliminate the effect of ground freezing on the shaft tower foundations, a special grid structure of foundations was peroposed with bases founded outside the effected zone of freezing Fig. 3. The foundation area of 1250 m^2 and 8,0 m depth has been designed in such a way, that its centre was left empty.

The space created in such a way, was used by mining works contractor, Mining Works Enterprise in Mysłowice, to install the freezing cellar as well as the brine freezing ring, dia 12,0 m and the safety platform at the depth of 8,0 m. Installation of the rope dead ends structures for the hanging platform have been designed as RC beams from the foundation grid above.

Excavation of the foundations was started as soon as freezing holes had been drilled, because of the complicated foundation shape as well as interpenetration of its particular sections. Final completion of works, connected with foundation construction wes proceeded by sinking down polyethylene pipes, dia 75 mm into the previously sunk freezing pipes, and assembly of the brine distribution ring. Having sunk them into holes after completion of the RC foundation roof plate, would have been impossible. After the above works had been completed, construction of shaft tower walls to the final height by means of slide method was passible. This was carried out at the end of the 1979.

Special sockets have been constructed in the walls and dummies have been installed in the RC during erection of overground tower structure, while driving the slide structure up. They have been used for installation of steel supports for shaft sinking.

Having completed the wall slide, an intermediate floor has been built to separate the upper and the lower parts of the tower.

The lower part, bolow+30 m level has been designed for installation of shaft sinking arrangements.

Completion of the upper part has been designed to be completed in parallel with the shaft sinking but assembly of permanent winding machines was to be carried out only after ground defrosting, in order to take into consideration any possible deviations of the shaft tower, resulting from settlement.

Platform for rope pulleys of winding machines, type C-3, 5×2A, winches ŁP-18/ 1000 for hanging platform and winches KU-BA-10 for steel shuttering of concrete lining has been designed in the lower part of tower among tower pylons at+30 m level. Winding machines for shaft sinking have been located at the southern part of the shaft tower on ground level, however winches ŁP–18/1000 have been placed at the western part on ground level too.

Winches KUBA-10 and cable winches have been mounted inside the tower in its pylons, at floor plate level.

The ropes of these winches were guided by means of a directing wheels system to the platform at +30 m level and from this platform down to the shaft.

Two chutes have been designed among tower pylons on the East-West tower axis. One for the eastern bucket and another for the western bucket.

In this case, sockets left in the lining of tower walls have been used during assembly of chutes structures.

4.
Freezing process and its control

Active ground freezing has been started after tower arection at the beginning of April 1980.

During ground freezing, systematic control of the freezing process has been carried out, basing on measurements of the brine temperature, actual time of freezing units operation and temperature measurements in thermal holes.

Determination of any irregularities of the freezing advance and its effects on the shaft tower foundation was the purpose of this control.

Analysis of the state of ground freezing around the shaft, especially in close-to-surface layers at the level of the tower RC foundations was carried out after every measurement had been taken. Thermal holes made possible determination of inside and outside isotherms of freezing mantle, and the thickness of the freezing mantle.

Results of temperature measurements in thermal holes, taken on 29th July 1980 and 22nd. August 1980 i.e. after four and four and a half months af active ground freezing, the thicknesses of the freezing mantle calculated are presented in Table No 1.

Measurements conducted on 29th of July 1980 have proved, that temperatures T_w in the inspection hole located at the distance of 6,0 m from the shaft centre line ranged from −14°C to −17° C/259°K− 256° K/. The thickness of the inner circle of the freezing mantle, calculated on the basis of these measurements, amounted to 2,5 –4,4 m. Temperatures T_z in the inspection bore-hole located at the distance of 8,75 m from the shaft centre line ranged from −1°C to −4°C /272°K −269°K/, and the calculated mantle thickness outside the circle of freezing holes was from 1,6 m to 2,0 m.

Thus, the total thickness of the freezing mantle amounted to 4,5 –5,0 m, i.e. it exceeded the designed thickness from 12 to 25%. Temperature measurements, taken on 22nd August 1980, in the outer thermal hole have shown further temperature drop to −3°C and −4°C /270°K −269° K/, and the increase of the freezing mantle around the shaft.

Analysis of the actual time of operation of freezing units as well as temperature measurements in thermal holes have also shown that in the period from April to August 1980 $1,17\times10^{13}$ J of heat has been carried away from the ground. exceedin the designed quantity by $1,14\times10^{13}$ J.

Basing on the above, it can be explicitly stated, that the planned state of ground freezing has been fully achieved, which allowed the commencement of shaft sinking by August 1980.

5.
The influence of ground freezing on the foundation of shaft tower

In order to determine the influence of ground freezing on the foundation of the shaft tower, four bench-marks were installed in the lower parts of walls in November 1979 /Fig. 3/. Measurements were taken with an 0,1 mm accuracy /Table No 2/. As the walls have been constructed, the shaft tower subsidence increased in proportion to the increasing weight and reached the biggest value of 5,8 mm under load amounting to 50% of total load /without. floors, foundations and machinery equipment/.

This subsidence resulted from consolidation and settlement of close-to-surface ground, especially sands and gravels. Active freezing started in April 1980, and despite placement of the foundations beyond the area of the frozen ring of ground, a slight upheaval of the shaft tower was caused reaching values similar to the initial state.

Measurements have been taken upto the time when shaft sinking reached below the freezing zone, i.e. 263 m and the final lining has been executed in this section.

The course of subsidence during the period of project realization is presented on Fig. 4.

Because bench;marks have been installed only on shaft tower walls, they had actually shown the vertical movements of the tower foundations, the bases of which have been located beyond the zone of influence of the freezing mantle. Lak of measurements of ground upheaval in the vicinity of the freezing pipes does not allow an estimate of how much less heave took place with the location of fundations away from the frozen zone.

Also, the lack of measurements of tower movement during ground defrosting does not give a full picture of final tower subsidence. Generally, it must be stated that besides the occurrence of slight ground deformations, being the result of external factors, their value was negligible and could have had no influence on the structure and stability of the shaft tower.

Conclusions and final remarks

1. In Polish coal mining, ground freezing under a permanent RC shaft tower, whichhas also been utilized for shaft sinking with bucket equipment, has been applied for the. first time.
   The construction of the RC shaft tower has been designed in such a way, that foundation bases have been located beyond the zone of influence of the frozen ground, whereas the interior of foundation remained empty, which made possible its utilization as a freezing cellar.
2. Adaptation of the shaft tower for the needs of shaft sinking allowed simultaneous carrying out of works connected with the assembly of final equipment and winding installations, as well as shaft sinking. It has considerably reduced the shaft construction cycle and speeded up the time of its commissioning.
3. Shaft tower subsidence, occurring during erection of walls, amounted to max. 5,8 mm. Such small subsidence was the result of good ground conditions of sands and gravels. The target subsidence may be considerable bigger, due to delayed even to few years settlement of less coherent grounds deposited below.
4. As a result of active freezing, foundation upheavals from 3,0 –5,0 mm have taken place, despite the location of foundation bases beyond the frozen ring area.
   This has occurred because of friction between the upheaving frozen ground and the not frozen rock masses.
5. Despite lack of a complex picture of the shaft tower subsidence it may be stated, that in this case, the process of tower construction, freezing and defrosting of ground as well as shaft sinking had no unfavourable influence on the shaft tower. However, it cannot be explicitly determined if in the case of presence of ground more liable to subsidence and more liable to upheavals during freezing/plastic formations/, the obtained results will be similar to the above mentioned ones.

Table No 1
Statement of freezing mantle thickness, calculated on the basis of temperature masurements in the inspection holes.

Depth	Temperature in thermal holes				Freezing mantle thickness$^{X/}$		
inner t_w		outer t_z		Ew	Ez	Rc	
	o_c		o_c		m	m	m

Measurements taken on 29 July 1980

Depth	Temperature in thermal holes						Freezing mantle thickness[X/]		
inner t_w			outer t_z			Ew	Ez	Rc	
	oc			oc			m	m	m
20	–		13,8	–		3,1	2,6	1,9	4,5
40	–		14,4	–		2,9	2 9	1,8	4,7
60	–		14,8	–		2,3	3,1	1,8	4,9
80	–		15,4	–		2,4	3,4	1,8	5,2
100	–		15,5	–		2,3	3,4	1,8	5,2
120	–		15,8	–		1,9	3,6	1,7	5,3
140	–		16,0	–		1,6	3 8	1,7	5,5
160	–		15,7	–		0,8	3,6	1,6	5,2
180	–		16,3	–		1,0	3,7	1,6	5,3
191	–		16,8		–		4,4	–	–
200		–		–		2,7	–	1,8	–
220		–		–		3,3	–	1,9	–
240		–		–		4,2	–	2,0	–
255		–		–		4,4	–	2,1	–
Measurements taken on 22 August 1980									
20		–			–	5,7	3,1	2,3	5,4
40		–				5,3	3,5	2,2	5,7
60		–			–	4,7	3,6	2,1	5,7
80					–	4,7	4,0	2,1	6,1
100					–	4,6	4,0	2,1	6,1
120					–	4,1	4,2	2,0	6,2
140					–	3,4	4,2	1,9	6,3
160					–	2,8	4,0	1,8	5,8
180					–	3,3	4,4	1,9	6,3
200					–	5,1	–	2,2	–
220					–	5,8	–	2,3	–
240					–	6,8	–	2,5	–
255					–	6,5	–	2,4	–

x/
Freezing mentle thickness calculated on the basis of temperatures in inspection holes.
E_w—inner freezing mantle thickness
E_z—outer freezing mantle thickness
R_c—total freezing mantle thickness

<u>Table No 2</u>
RESULTS OF GROUND /VERTICAL/ DEFORMATIONS

Sl. No	Date of measurement	Levelling								
RP 1		RP 2		Rp 3		RP 4				
Spot level / m/	Difference / mm/	Spot level / m/	Difference / mm/	Spot Level / m/	Difference / mm/	Spot level / m/	Difference / mm/			
1	2	3	4	5	6	7	8	9	10	11
1	27.11.1979	243,8701		243,8778		243,9107		243.8487		
2	4.12.1979	243,8700	0,1	243,8776	0,2	243,9105	0,1	243,8487	0,0	
3	11.12.1979	243,8701	0,0	243,8776	0,2	243,9107	0,0	243,8487	0,0	
4	19.12.1979	243,8685	1,6	243,8761	1,7	243,9093	1,4	243,8481	1,6	
5	2.01.1980	243,8689	1,2	243,8764	1,4	243,9090	1,7	243,8475	1,2	

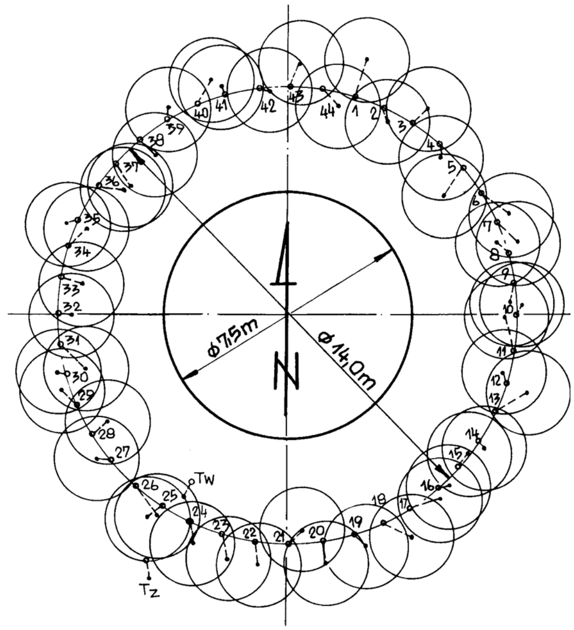

Fig.1 Freezing hole deviations at-250m level shaf t no. II

Sl. No	Date of measuremen t	Levelling								
RP 1		RP 2		Rp 3		RP 4				
Spot level / m/	Difference / mm/	Spot level / m/	Difference / mm/	Spot Level / m/	Difference / mm/	Spot level / m/	Difference / mm/			
1	2	3	4	5	6	7	8	9	10	11
6	12.01.1980	243,8683	1,8	243,8754	2,4	243,9086	2,1	243,8469	1,8	
7	21.01.1980	243,8676	2,5	243,8758	2,0	243,9082	2,5	243,8463	2,4	
8	1.02.1980	243,8670	3,1	243,8750	2,8	243,9078	2,9	243,8457	3,0	
9	12.02.1980	243,8664	3,7	243,8742	3,6	243,9072	3,5	243,8450	3,7	
10	20.02.1980	243,8659	4,2	243,8733	4,5	243,9068	3,9	243,8443	4,4	
11	27.02.1980	243,8654	4,7	243,8727	5,1	243,9065	4,2	243,8439	4,8	
12	5.03.1980	243,8652	4,9	243,8720	5,8	243,9061	4,6	243,8435	5,2	
13	26.07.1980	243.8668	3,3	243,8727	5,1	243,9083	2,4	243,8456	3,1	
14	1.08.1980	243,8668	3,3	243,8728	5,0	243,9988	1,9	243,8456	3,1	
15	9.08.1980	243,8674	2,7	243,8743	3,5	243,9095	1,2	243,8457	3,0	
16	25.08.1980	243,8679	2,2	243,8745	3,3	243,9095	1,2	243,8459	2,8	
17	25.09.1980	243,8685	1,6	243,8753	2,5	243,9101	0,6	243,8466	2,1	
18	20.12.1980	243,8681	2,0	243,8750	2,8	243,9099	0,8	243,8468	1,9	
19	20.01.1981	243,8689	1,2	243,8754	2,4	243,9108	+0,1	243,8473	1,4	
20	20.02.1981	243,8690	1,1	243,8750	2,8	243,9108	+0,1	243,8475	1,2	
21	20.03.1981	243,8698	0,3	243,8750	2,8	243,9112	+0,5	243,8477	1,0	
22	20.04.1981	243,8702	+0,1	243,8755	2,3	243,9107	0,0	243,8476	1,1	

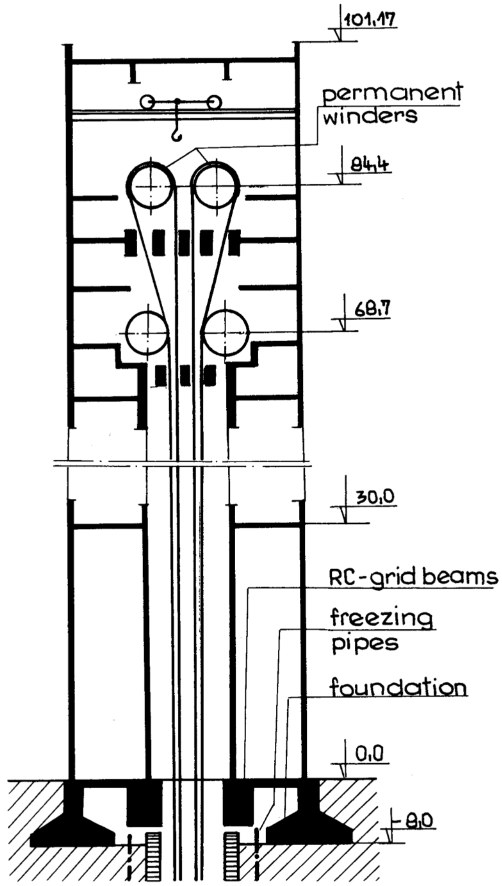

Fig.2 Reinforced concrete hoisting tower constructed before commencement of shaft sinking by application of the ground freezing method —vertical cross-section

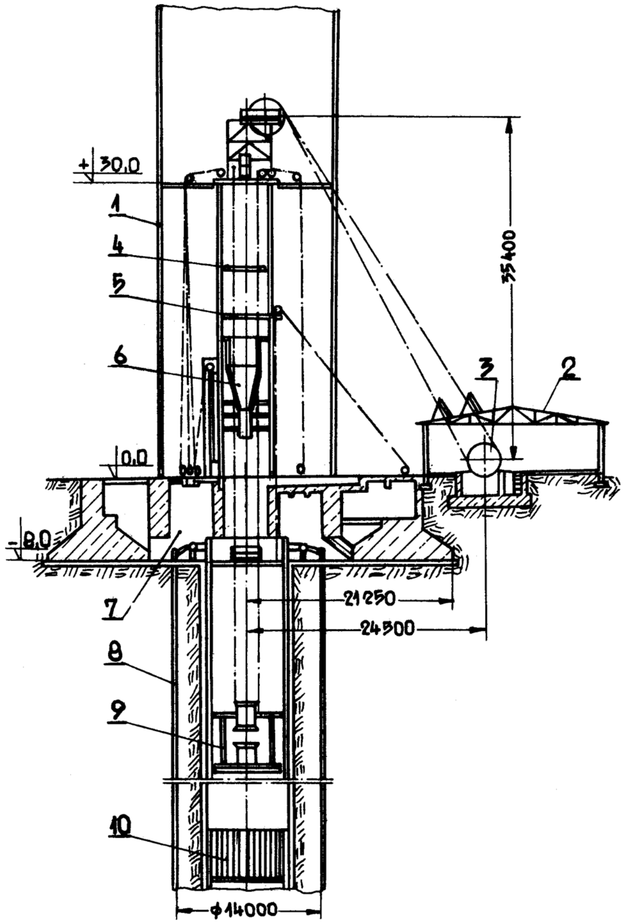

Fig.3 Vertical cross-section of shaft no.II and location of winders

4–permanent RC hoisting tower

2–winder building

3–winders C-3,5×2A

4–bucket catcher deck

5–signalman deck

6–chute

7–freezing cellar

8–freezing holes

9–scaffold

10–steel shuttering

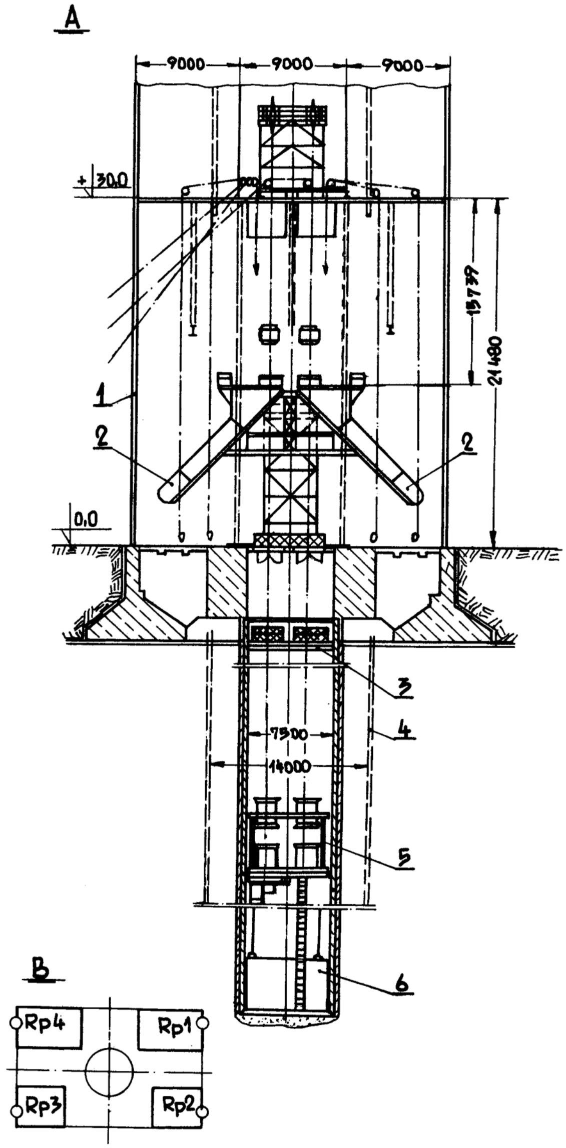

Fig.4 A.Layout of shaft sinking equipment shaft no.II

4–Reinforced Concrete hoisting tower

2–chute

3–safety platform

4–freezing hole

5–scaffold

6–steel shuttering

B. Location of bench-marks

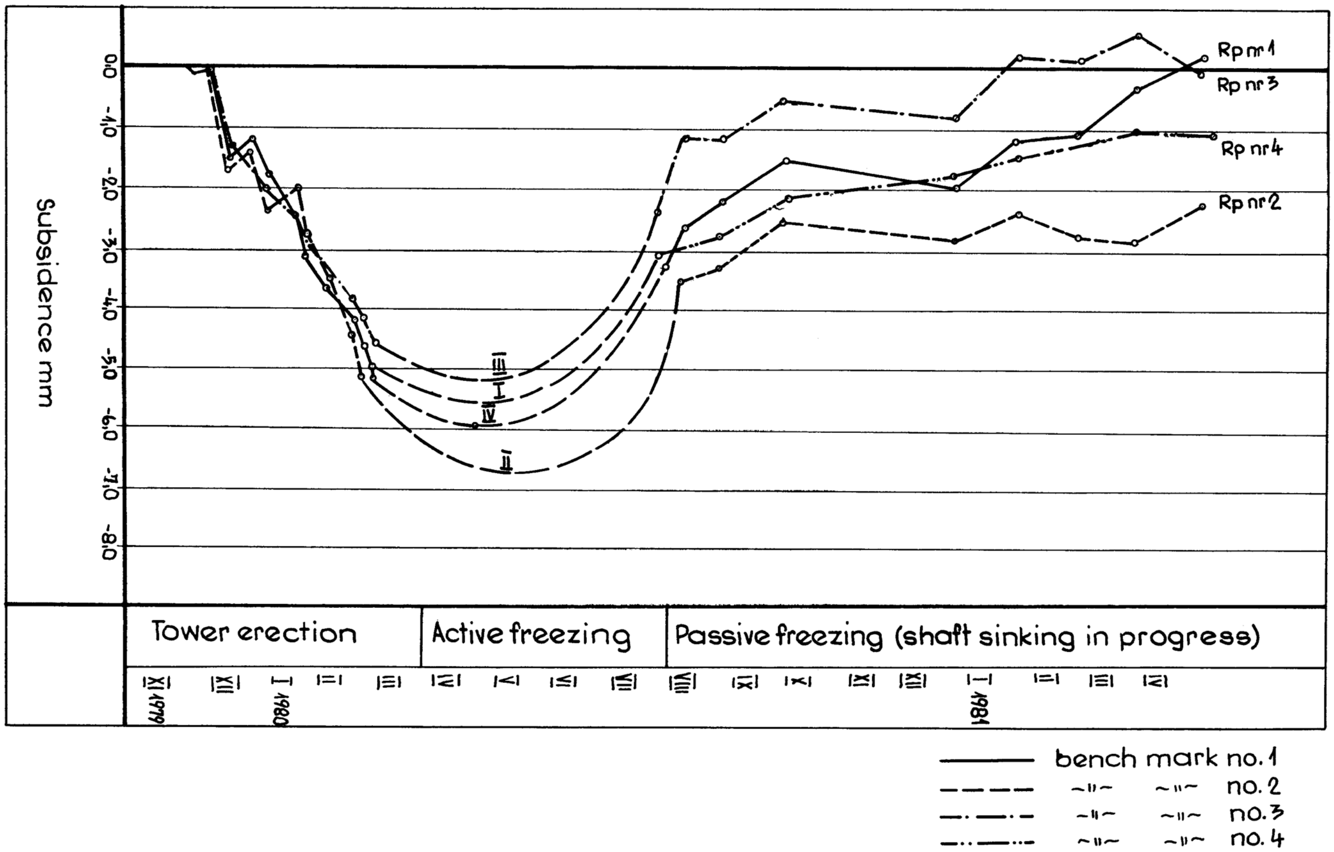

Fig. 5 HOIST TOWER SUBSIDENCE DIAGRAM

Design of unlined pressure shafts

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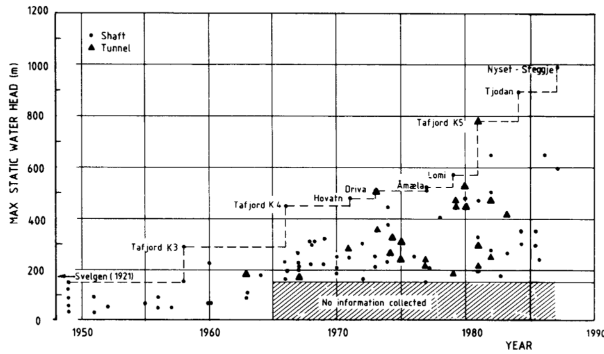

Fig. 1 Development of unlined pressure tunnels and shafts, after ref. 6

SUMMARY

The evolution in design practice for unlined pressure shafts is described. The present practice where a FEM model is used for preliminary design and in situ stress measurements during the construction phase for verification is discussed.

The necessity of doing additional testing in areas with complex geology and hydrogeology is stressed. High leakage losses due to permeable ground and low phreatic level may have serious impact on project economy.

The procedures are illustrated by examples described separatly in appendix 1, 2 and 3. The unlined shafts at Tjodan, head 880 m and NysetStegje 1000 m are described, as is the 700 m head shaft at Alfalfal in Chile where steel lining was recommended.

INTRODUCTION

The reason for using unlined pressure conduits in hydropower development is purely economical. At the current Norwegian price level the ratio between costs for a concrete embedded steel lined shaft and an unlined shaft is approximately 3:1.

Very often it is also the case that a reduction of lined length leads to a reduction of total construction time. Both factors contribute to project economy and may tip the balance on marginal projects.

This is and has been a rather selfevident fact, but the reliability of and the confidence in the design basis has called for a gradual development.

The fig. 1 illustrates this slow process which also is lined by some failures.

The failures reported are seemingly all cases of water pressure exceeding virgin stresses in the rock mass[2,3,6]. Reports from Austria[5], from Colombia[13] and on two North American failures[1] underlines the importance the stress situation in the design of unlined pressure conduits.

Where permability of the rock mass is concerned, the published material seems to focus on how to stop leakages. The problem of designing a tunnel in a permeable material where permanent loss of water is considered, is not widely treated [4, 10,11]

Leakages measured after first filling have been basis for calculation of large scale rock mass permeability. Testing in drillholes and measurement of leakage into tunnels have been used for prediction of loss during operation, [8,9].

For the Osa power plant in Norway, the sealing works undertaken there after about 1 year of operation is described. The cost and benefit of sealing is discussed[14].

FIELD INVESTIGATIONS

As a part of the design procedure, detailed engineering geological investigations have to be performed to decide whether the conditions are favourable or not regarding an unlined pressure conduit.

A number of geological factors which are of major importance for design and construction of unlined pressure shafts have to be investigated.

These rock mass properties are:

- Type of rock
- Rock structure including jointing and faulting
- Permeability
- Position of phreatic level
- State of stress
- Physical and chemical stability
- Seismic activity.

General geological mapping is the basis for a detailed field investigation program. In this investigation program the rock type, joint orientation and frequency is mapped.

Special care is taken to identify pervious zones and fractured zones which might have induced stress release or stress concentration in the rock mass.

In cases where the internal pressure in the tunnel is higher than the groundwater table, drainage of water from the tunnel may have environmental impact. The possibility of sliding caused by saturation should therefore be considered. Hydrogeological mapping and especially the location of the groundwater table are essential.

It is also important to investigate the seismic activity of the area. As a principle a pressure shaft must be located in a stable geological unit without intersecting active fault zones.

As mentioned in the introduction, the in situ stresses in the rock mass are very important. The main requirement is that the miniumum principal stress should be higher than the water pressure at every point along the shaft or tunnel to avoid the risk of failure by hydraulic fracturing or jacking.

In areas where the geology and the regional in situ stress situation is known from former measurements, it is usual in Norway at an early stage of a project to base the design on overburden criterias or design charts based on FEM analysis. In this case it is necessary to verify the assumptions by performing stress measurements from the tunnel during construction.

If, however, a pressure tunnel or shaft is planned in an area where the geological and tectonic situation is complex, core drillings have to be started in critical areas along the tunnel or along the planned shaft axis even at the feasability stage.

The drillings will give important and necessary geological and hydrogeological informations. It is also possible to do stress measurements in the boreholes, even in deep boreholes.

The most frequent stress measurement technique used in deep boreholes is the hydraulic fracturing method. Rock fracturing in the holewall is initiated by pumping water in between a packer and the bottom of the borehole or between straddle packers at various depths in the hole.

Based on informations from engineering geological and hydrogeological mapping, test drilling, an appraisal of the rock stress situation and in some cases stress measurements a decision on whether a high pressure conduit is feasible or not, can be taken.

DESIGN PROCEDURE

General

From the previous chapter the following geological factors may be regarded as main criterias for the pressure shaft design procedure:

- Low permeability of the rock material
- Low permeability of joints and factures
- Virgin stress sufficiently high to prevent fracture or jacking on joints
- Physically and chemically stable rock masses.

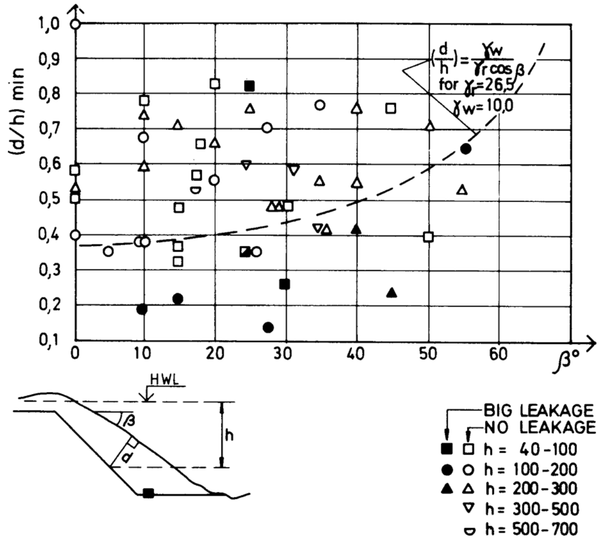

Fig. 2 Overburden criteria as function of inclination, β° of valley side, after ref. 8

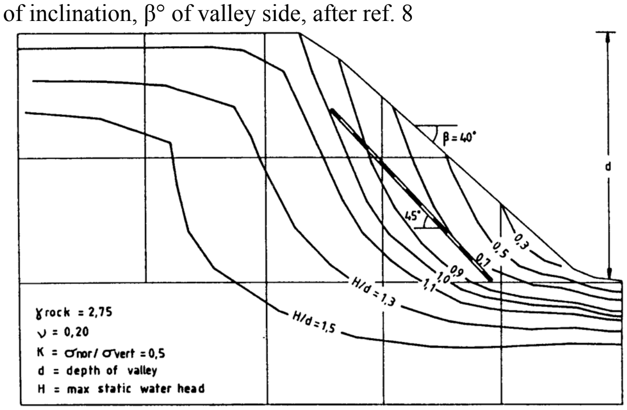

Fig. 3 Design chart based on FEM analysis, after ref. 12.

The only condition which may be termed absolute is the stress requirement. Permeability and durability may be increased by suitable treatment of the rock mass. These problems are more cases of project economy and optimization— provided that it is possible to quantify the amount of work involved.

Even a stress deficiency of a local nature may be overcome provided that the rock mass modulus of deformation is high enough to allow a prestressing by pressure grouting.

Development of design criterias and practice

Design of pressure conduits was done long before reliable measurement techniques were developed and accepted. Hence the design with respect to virgin stresses had to be based on theoretical considerations and certain assumptions.

Several simple overburden criterias have been developed.[3,15,16]. The necessary requirement for a stable tunnel location was that the water pressure should be less than the minimum principal rock stress. The assumption was that this stress was the vertical stress component or a stress component with some angle to the vertical depending both on surface slope or shaft inclination.

An overburden criteria developed by Bergh-Christensen and Dannevig has found wide use[17]. Data from various pressure shafts and tunnels are compared to this criteria in fig. 2.

As evident from fig. 2, the criteria seems to cover the majority of cases, but at least one failure cannot be explained. Even if used as recommended by several authors, showing a caution where topographic features like ridges and protrutions in vally sides are concerned, the criteria is clearly not suitable for final design. It is, however, a handy tool for project evaluation at a concept stage.

A finite element model was developed[2] and later complemented by design charts, fig. 3.

The design charts ment that a simple and fairly correct stability analysis could be done. Most of the unlined pressure conduits designed in the seventies and early eighties, see fig. 1, were designed with the help of the design charts.

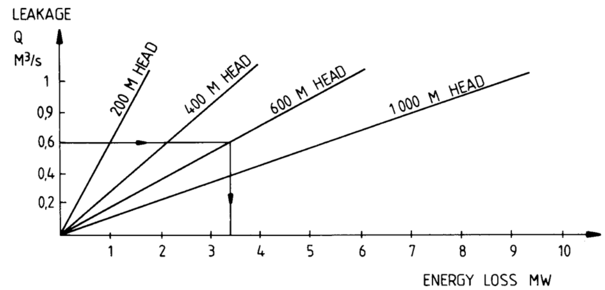

Fig. 4 Energy loss as function of leakage and static head.

One limitation of the design charts in their present form is that they are based on a situation with high horizontal stresses. This is the typical case for the Baltic and Canadian shields, but is not necessarily a general fact. Cases with varying horizontal tectonic stresses can easily be modelled and sensitivity for variations in for example Young's and Poisson's ratio may be analysed. Other limitations may be more important. A planar model is based on the assumption that the critical minimum principal stress occurs in the plane and not perpendicular to it or parallel to a vally side.

The analytic tool also makes a certain idealization of topography necessary. The profile must be smothed and as for the overburden criteria the effects of ridges and protrutions have to be considered.

Notwithstanding the limitations, the FEM analysis is useful. Depending on special project requirements the FEM analysis may be used even for the design which is the basis for bid documents. It is then necessary to provide options for minor adjustments of shaft location at the construction stage without negative effects on costs and schedule. This is in fact now a common approach in Norway. A preliminary location of the shaft is done based on FEM models and provisions for relocation are included in the contract.

A difficult part of pressure shaft design is the evaluation of potential leakage.

Parameters like position of groundwater table relative to a pressure shaft, general rock mass permeability and the existense of especially pervious zones are factors which must be considered.

Figure 4 shows the approximate energy loss for combinations of leakage and static head.

The figure illustrates the possible impact of an underestimated leakage and clearly calls for a conservative design.

The majority of the earlier Norwegian pressure shafts were designed based on the assumption that the rock mass permeability was so low that the ensuing leakage was of no practical importance. This seems to be correct for the Paleozoic and older rock types of the Baltic shield insofar as all reports on problematic leakage are clear cases of hydraulic fracturing or hydraulic jacking.

Calculations of permeability based on leakage into tunnels during construction and leakage from the tunnels have been done for some Norwegian tunnels 8.

The results are shown in table 1.

TABLE 1. Summary of layout, geology and leakage control results from six hydro power plants, after ref. 8

POWER PLANT		MAX HEAD ON UNLINED ROCK IN BAR	LAYOUT	GOLOGY	PREDICTED LEAKAGE IN $1 \cdot s^{-1}$	MEASURED LEAKAGE IN $1 \cdot s^{-1}$	CALCULATED MASS PERMEABILIT Y IN $m \cdot s^{-1}$
JØRUNDLAND	1971	28	2.0 km pressure tunnel	precambrian granite and gneiss	–	1	$1 \cdot 10^{-9}$
SKJOMEN	1973	36	2.6 km pressure tunnel	precambrian granite	–	1–2	$3 \cdot 10^{-9}$
BORGRUN	1974	25	2.9 km pressure tunnel	precambrian gneiss	–	3–4	$1 \cdot 10^{-8}$
LEIRDØLA	1978	45	0.6 km pressure shaft	precambrian gneiss	–	0.9	$1 \cdot 10^{-8}$
LOMI	1979	59	0.7 km pressure shaft	ordovician and phyllite	1–5	3–6	$5 \cdot 10^{-8}$

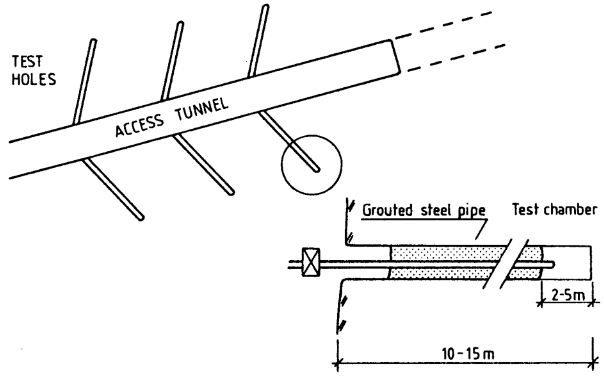

Fig. 5 Typical arrangement of hydrofracture test holes and hole detail

POWER PLANT		MAX HEAD ON UNLINED ROCK IN BAR	LAYOUT	GOLOGY	PREDICTED LEAKAGE IN $1 \cdot s^{-1}$	MEASURED LEAKAGE IN $1 \cdot s^{-1}$	CALCULATED MASS PERMEABILIT Y IN $m \cdot s^{-1}$
SKIBOTN	1980	44	4.0 km pressure tunnel	ordovician mica schist	2–10	10–18	$3 \cdot 10^{-8}$

The calculations done in table 1 based on Eq (1) confirm a general low rock mass permeability.

$$Q = \frac{k \times 2\pi \times L \times \Delta P}{\ln D/r} \tag{1}$$

k	=	Rock mass permeability (ms^{-1})
L	=	Length (m)
Δ P	=	Net head (m)
D	=	Distance to equipotensial (m)
r	=	tunnel radius (m)

The Eq (1) shows that the most important factor when leakage is concerned is net head. This means that in areas with low phreatic level the leakage potential is high. In areas like the young and tectonically active mountain ranges of Asia and America conditions may deviate consider ably from the environment on which the previous experience is based.

Steep mountains, complex in situ stresses and low precipitation may result in unfavourable conditions.

Under such conditions great care should be taken in mapping the groundwater level. High pressure conduits should be investigated by drilling and testing at the feasability stage of planning and should be located under "groundwater cover" where possible. In evalutation of test results from such drillings a probable case and a worst case should be analysed with respect to leakage. The lead time on steel-lining delivery is long and the consequences of an underestimated leakage is therefore considerable.

VERIFICATION MEASUREMENTS AT THE CONSTRUCTION STAGE

For the final design of the pressure shaft or tunnel it is very important to verify that the assumed stresses are similiar to the stress conditions in situ. When the tunnelling works begin, in situ stress measurements are usually performed at a location as close to the shaft as possible while still maintaining the possibility of design modifications.

As the required input for final design is the minimum principal stress, a simple hydraulic fracturing or jacking tests is used. The hydraulic jacking in a drillhole is a scaled model of the pressure conduit itself. The tested area is in the order of magnitude of meter squared. A limited number of tests will therefore yield a representative average value.

A practice of testing in 10–15 m long holes has developed in Norway. Holes drilled by the tunnel jumbo is prepared for testing by grouting a steel pipe in place. In reasonably tight rock mass a chamber lengths of 2–5m have been used, see fig. 5.

Figure 6 shows pressure versus time curves for a hydraulic fracture test.

The practice of taking the shut in pressure as minimum principal stress has gained general acceptance. It could be argued that for pressure tunnels the fracture opening pressure indicated on fig. 6, 2nd cycle should be used for design.

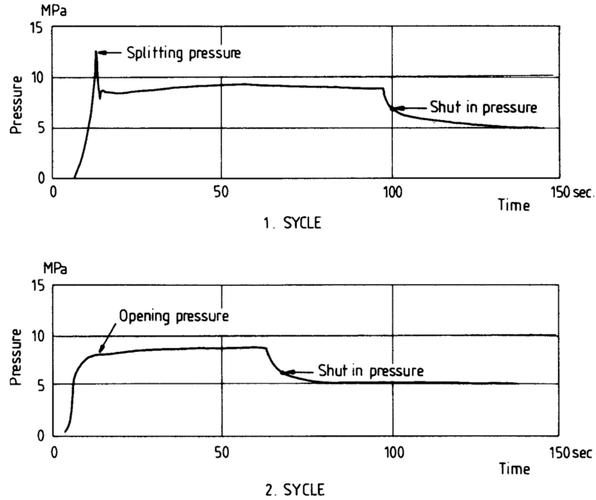

Fig. 6. Typical plot for a hydraulic fracture and jacking tests.

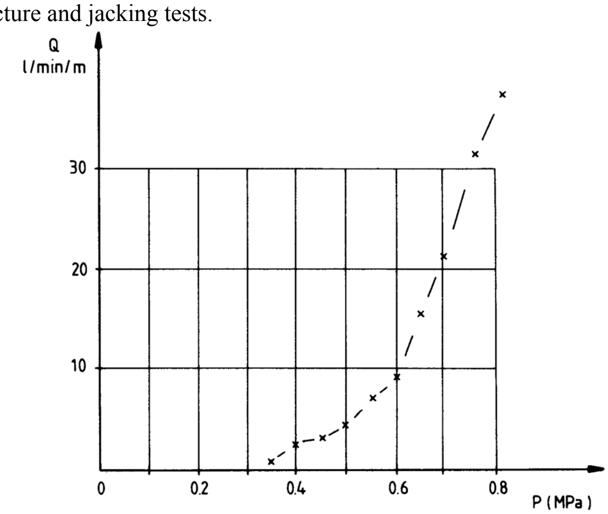

Fig. 7. Hydraulic jacking in a modified LUGEON test.

At the Alfalfal project in Chile and the Mayush project in Peru hydraulic jacking tests were conducted as modified Lugeon tests. Figure 7 shows a typical plot from these tests indicating a gradual opening of joints over a certain pressure range. This means that significant increase in leakage may occur before a full opening of joints or fractures takes place.

The broad opening range shown on fig. 7 indicates that the more conservative shut in pressure is the best basis for pressure shaft design.

Stress measurements based on the overcoring technique are also used. This method is often more expensive and timeconsuming than hydraulic fracturing as laboratory testing is involved. The resolution of the instruments used also has an influence on the accuracy. The accuracy may be ±1−2 MPa which is not always satisfactory in the stress range which is interesting for unlined pressure shafts.

CONCLUSION

The design practice for unlined pressure shafts has developed gradually to its present state. Overburden criteria of finite element models are used for preliminary shaft design. A verification of design parameters in situ is usually done. As a rule this is obtained by doing hydraulic splitting tests from a convenient access tunnel close to where the excavation of the shaft is planned. The cruical parameter measured is minimum principal stress.

The construction contract should provide for possible minor changes in shaft location or in lay-out if the stress measurements demonstrate the advisability of this.

Where complex geology or hydrogeology occur, both permeability and in situ stress should be checked at an earlier stage in the design process. Low groundwater level or high permeability may render an unlined shaft unfeasible even if the in situ stresses are sufficient for stability.

Evaluation of leakage from the shaft, the energy loss represented by the leakage and its value should be done. In view of the lack of precission in the input data, namely rock mass permea-bility and gradient, a conservative approach should be used.

AKNOWLEDGEMENTS

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References:

1. Terzaghi, K. (1953) Unpublished notes on failure of Sand Creek Siphon and Watshan Power Station, The Terzaghi Liberary, N.G.I., Oslo
2. Brekke. T.L. et al (1969) "Finite element analysis of the Byrte unlined pressure shaft failure" Proc. of Symp. on Large Permanent underground Openings, Oslo
3. Selmer-Olsen, R. (1969) "Experience with unlined pressure shafts in Norway" Proc. of Symp. on Large Permanent Underground Openings, Oslo
4. Barton, N. (1972) "Estimation of leakage rate and transport time for fluid flow from underground openings in jointed rock" NGI report 54203–1, Oslo
5. Seeber, G. et al (1979) "Die Schäden im Hattelberg—druckstollen als folge eines ausergewöhnlichen Primär-spannungs Zustandes" Proc. 4th ISRM conf., Montreux
6. Broch, E. (1982) "Development of unlined pressure shafts and tunnels in Norway". Proc. intnl. Symp. Aachen
7. Bergh-Christensen, J. & Kjølberg, R. (1982) "Investigations for a 1000 meter head unlined pressure shaft at the Nyset/Steggje Project, Norway" Proc. intnl. Symp. Aachen
8. Buen, B. & Palmstrøm, A. (1982) "Design and supervision of unlined hydropower shafts and tunnels with head up to 590 meters" Proc. intnl. Symp. Achen
9. Johansen, P.M. & Vik, G. (1982) "Prediction of air leakage from air cushion surge chambers" Proc. intnl. Symp. Achen
10. Tokheim, 0. & Janbu, N. (1982) "Flow rates of air and water from caverns in soil and rock Proc. intnl. Symp. Achen
11. Schleiss, A. (1986) "Design of pervious pressure tunnels" Water Power and Dam Construction, May, p. 21–26, 29
12. Palmstrøm, A. (1987) "Norwegian design and construction experience of unlined pressure shafts and tunnels." Proc. intnl. Symp. Underground Hydropower Plants, Oslo
13. Broch, E. (1985) "Unlined high pressure tunnels in areas of complex topography" Pub. no 3. Norwegian Soil and Rock Eng. Ass. Trondheim
14. Edvardson, S. & Sæteren, B. (1985) "Excavation and leakage control of unlined air cushion surge chamber at the Osa hydroelectric project" Pub. no 3. Norwegian Soil and rock eng. ass. Trondheim
15. Kieser, A. (1960) "Druckstollenbau" Springer Verlag, Wien
16. Terzaghi, K. (1963) "Stability of steep slopes on hard unweathered rock" Publ. no 50, NGI, Oslo
17. Bergh-Christensen, J.Dannevig, N. (1971) "Design of unlined pressure shaft at Mauranger" (in Norwegian) Geoteam A/S report 2398.03, Oslo

APPENDIX 1
PRESSURE SHAFT DESIGN AT THE NYSET-STEGGJE POWER PLANT

This power project in Årdal, Norway includes a 1130 m long unlined pressure shaft with a static head of approximately 1000 m.

From the reservoir the water is fed through a headrace tunnel to the top of the pressure shaft at an altitude of 800 m. The pressure shaft itself has an inclination of 45° with its lower end (at sea level) just inside the power plant.

The terrain in the shaft area consists of a high altitude plane at el. 1100–1200 m with steep irregular rocky slopes down towards the fjord and the valley along the Nyset river, fig. 2.

The rocks in the shaft area consit of gneiss at the bottom along the Nyseth River. The gneiss is overlain by an intruded white granite. The boundary between these two rocks is very irregular. Originally the boundary zone has been crushed, but because of injection of granitic material it is now massive and sound. The thickness of this zone is about 200–250 m.

The location of the shaft was based on a calculation of total stresses by finite element plan strain analysis. In this case the terrain is very irregular, and therefore it was necessary to establish a simplified topographic model where all the protruding noses were cut away. The simplified model of the terrain is shown in fig. 3.

The computer programme used gives the stress distribution along a vertical plan normal to the contour lines of the terrain model. In this case the shaft had to be located obliquely to the valley side and therefore the standard programme did not give the stress distribution in a single run.

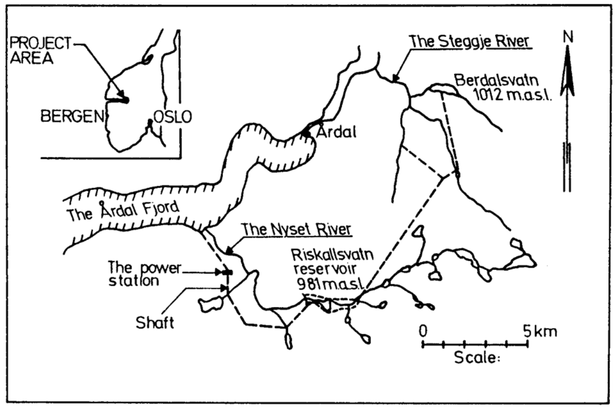

Fig. 1. The Nyset-Steggje hydropower project

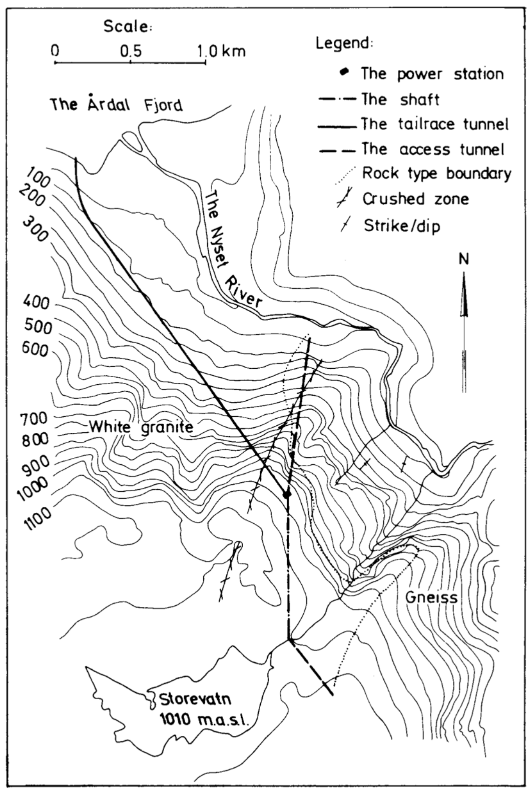

Fig. 2.

A three-dimensional model was found unrealistic and calculations were performed with the use of three two-dimensional models representing the profiles A, B and C cutting the shaft at 425 m above the bottom (A), at 225 m above the bottom (B) and at the bottom of the shaf t (C). The geometry of one of the models is shown in figure 4. Each model is divided in sub-structures and the shaft is located in sub structure 1.

Fig. 5 shows the calculated minimum total stresses at the intersection points between the shaft and the sections A-A, B-B and C-C. The corresponding water pressure is also shown.

The following values for the ratio between minimum total stress and water pressure was found:

It was concluded that the preliminary location of the shaft was sufficiently safe, and this location was chosen for the final design.

When the tunnelling works started, the access tunnel to the power plant and a pilot adit towards the bottom of the shaft had the highest priority.

Rock stress measurements were performed in the access tunnel, in the powerstation area and in the tunnel between the station and the bottom of the shaft. Totally 4 stress measurements by the overcoring method and 9 hydraulic fracturing tests were performed.

Figure 6 shows the results from the hydraulic tests in the powerstation—sandtrap area.

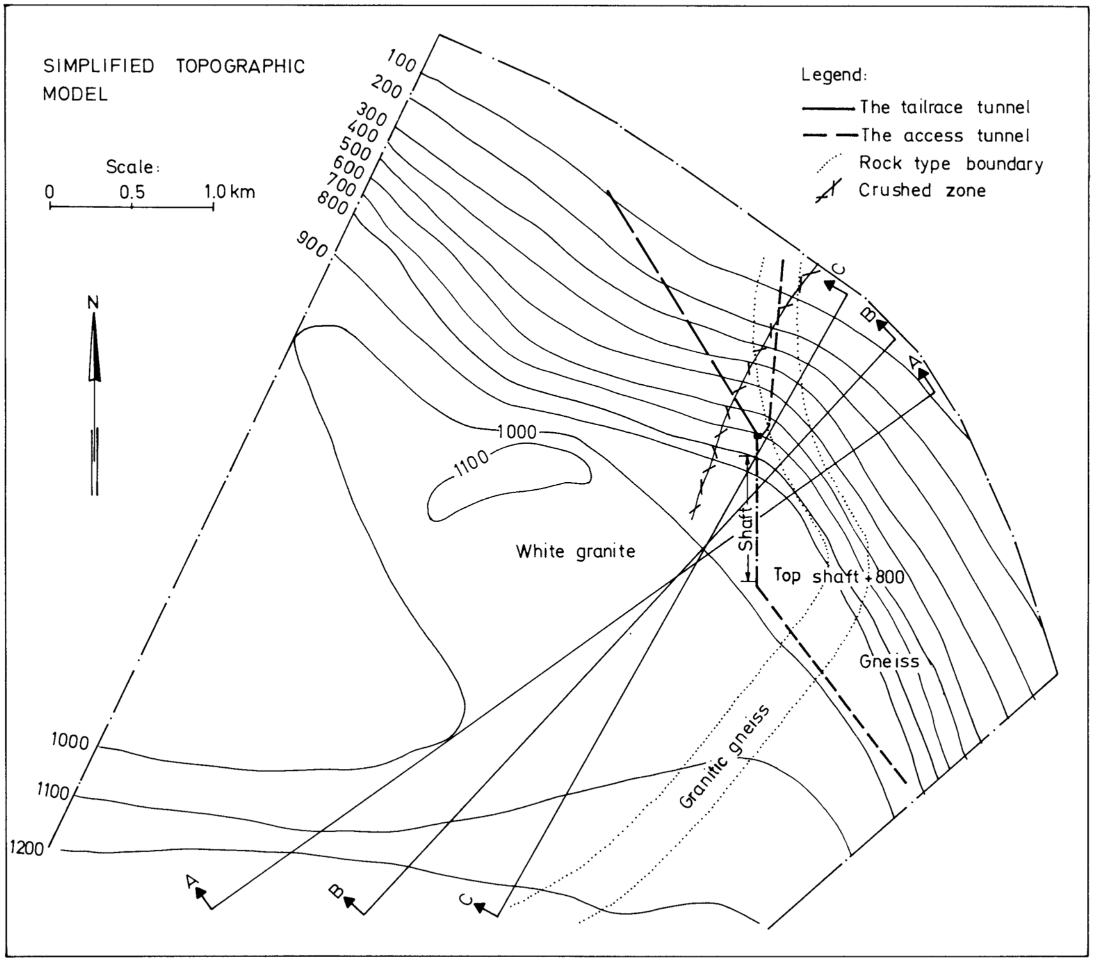

Fig. 3.

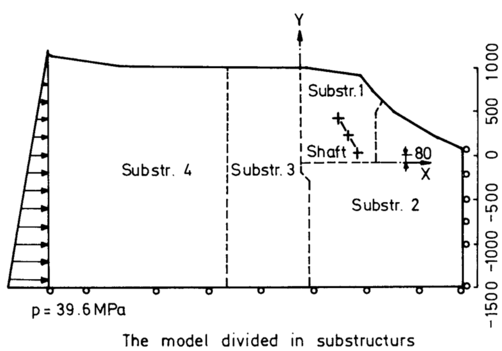

Fig. 4.

The lowest measured value of the shut in pressure in hole 7 is 12.3 MPa. Based on this value the ratio between the minimum principal stress and the water pressure is 12.3/9.64, which gives a safety factor of 1.3 against hydraulic fracturing.

If this value of the safety factor is compared with the calculations done at the planning stage, it will be found that this is the same value as given by the overburden criteria-method introduced by Bergh Christensen and Dannevig (ref. 17 and fig. 2 in the main paper), while the finite element analysis predicted a higher value, 1.5.

As a conclusion it was found that the location calculated on the planning stage gave a satisfactory safety against hydraulic fracturing and a fairly good accordance between the assumptions done at the planning phase and the conditions verified in situ.

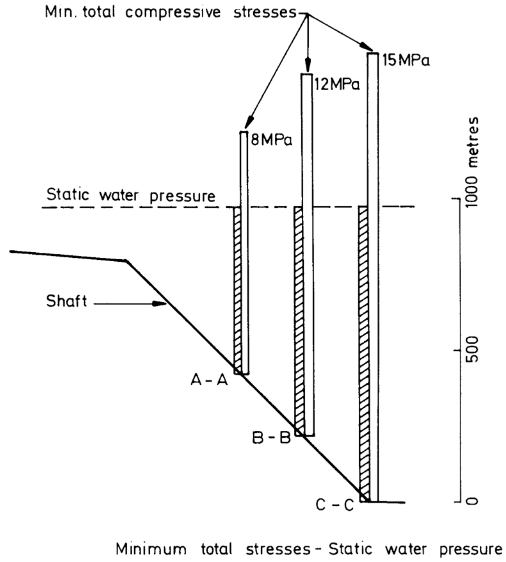

Fig. 5.

Section	A–A	:	1.46
"	B–B	:	1.58
"	C–C	:	1.52

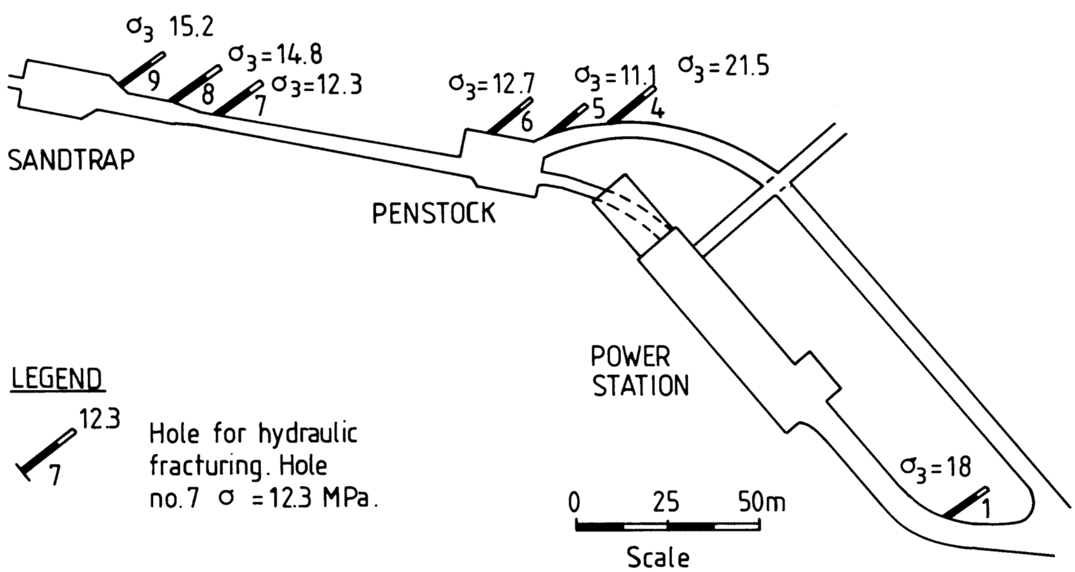

Fig. 6 Hydraulic splitting tests in the power station area, Bergh-Christensen (1985)

References:

Bergh Christensen, J. & Kjølberg, R. (1982) "Investigations for Norway longest unlined pressure shaft". Water Power & Dam Construction, April 1982

Bergh Christensen, J. (1985) "Design of unlined pressure shaft at the Nyset-Steggje Hydropower Project" (in Norwegian). Fjellspr.tekn. bergmek. 1985. Tapir.

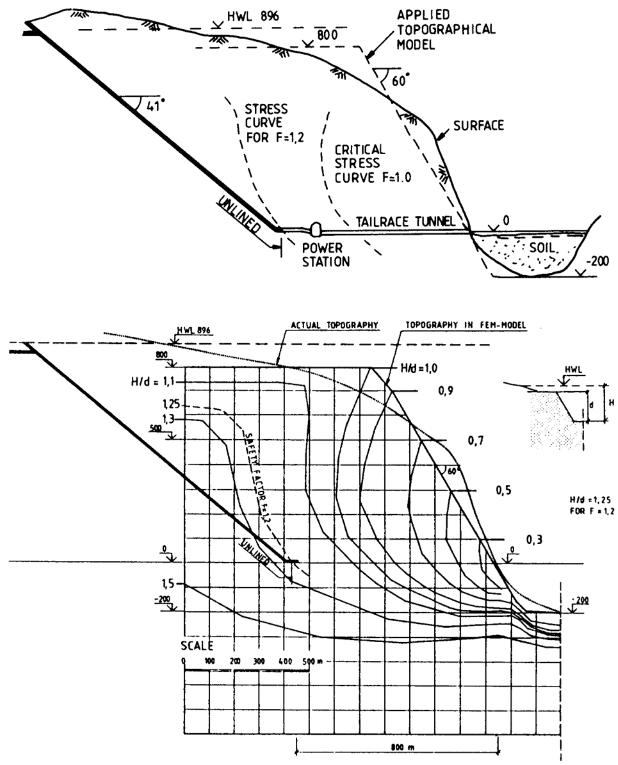

Fig. 1 Sections showing terrain profile and the FEM model applied

APPENDIX 2
PRESSURE SHAFT DESIGN AT THE TJODAN PROJECT

The Tjodan plant is situated in southeast Norway, some 50 km east of the town of Stavanger, The power plant has an output of 110 MW from a single Pelton turbine with a head of 900 m and a production of 310 GWh/year. The drainage area is 54 km2 and the precipitation is approximately 3000 mm/year.

The powerplant is a typical example of current Norwegian hydropower construction, consisting of 10 km of tunnels collecting water through five reservoirs and five secondary intakes from a wide area and utilizing it in a 900 m head Pelton turbine.

The Power plant was commissioned in 1984.

The shaft, 1250 m long with diameter 3.5 m was excavated on 41° inclination by TBM. The shaft is situated in massive precambrian gneiss.

The geological mapping of the area showed that the gneisses had a low to moderate degree of jointing. No weakness or fracture zones of importance were found which would interesect the planned shaft. Laboratory tests of the gneisses showed an average compressive strength of 160 MPa and a Poisson's ratio of 0.2.

To assess the overburden required, a simple FEM calculation was first performed with standard diagrams. for this purpose the terrain in a section along the length of the shaft was simplified (Fig. 1). With a selected safety factor of 1.2 the power station was preliminarily located 800 m inside the mountain.

To provide a more accurate picture of the probable magnitude of the minimum principal rock stress, a special FEM analysis was performed. The computations carried out were adapted to the topographical and geological conditions met with at Tjodan. Five different models of the terrain were used for this purpose. These indicated that the pressure shaft had a probable safety factor of 1.2–1.35, depending on the different assumptions of rock stress distribution. The preliminary location was therefore found acceptable for detailed design.

In order to verify the calculated in situ stresses two types of in situ measurements were done. These consisted of both three-dimensional over-coring stress measurements and hydraulic splitting tests in the powerhouse area. The measurements were first performed in the adit down to the tailrace tunnel 150 m from the station (Fig. 2). The reason for carrying out the first stress measurements here was to be able to change the planned location of the power station if there proved to be an appreciable difference between stresses actually measured and those computed.

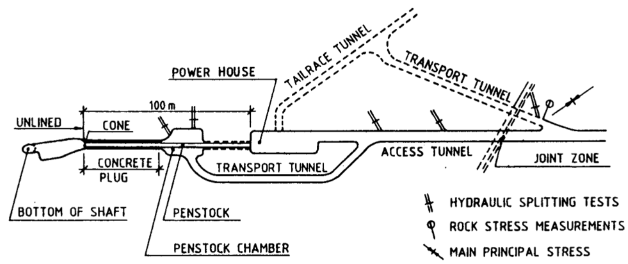

Fig. 2 Locations of rock stress measurements performed in powerhouse area.

The survey showed that the directions of the principal stresses were more or less what had been expected, with the main principal stress running just about parallel to the steepest portion of the mountainside. The magnitude of the stresses was, however, less than had been estimated, as the stresses appeared to be contingent only upon the weight of the overlying rock, whereas included in the FEM analysis were factors pertaining to the horizontal stresses. The results of the hydraulic splitting tests, likewise, showed low stresses.

When the access tunnel was driven a litle further it was found that there was a joint zone just inside the measuring point.

It was assumed that this feature affected the rock stress pattern. The tendency towards spalling phenomena inside the zone supported this assumption. Accordingly, it was decided to rely more on the hydraulic splitting tests to be performed in the penstock chamber inside the zone (see Fig. 2) before the final location of the pressure shaft was decided.

The results here showed that the minimum principal stress was about 12.5 MPa, 0.6–2 MPa more than had been estimated in the FEM calculations. With the same location as had been preliminarily chosen, this gave a minimum safety factor of 1.43 against hydraulic splitting for the shaft. This was accepted as satisfactory and the preliminary location was maintained for construction.

While the shaft was being TBM drilled, both water leakages and geological conditions were continuously monitored. On the basis of the data thus compiled, the rock support and sealing measures needed were planned and described. At five places extensive sealing and grouting work had to be carried out.

While a shaft is being excavated, joints and pores in the surrounding rocks are drained to the point that leakage, fed by the groundwater in the rock mass, is reduced to an even flow. For Tjodan this was recorded as 0.2 m3/min into the entire shaft.

When a shaft is filled with water the joints, and pores in the drained zones around the shaft become filled to. Extreme local gradients and stresses may result in deformation if the filling is rapid.

This hazard can be reduced substantially by slowly filling the shaft.

If the first filling of the shaft is done slowly and sequentially, extensive, unforeseen leakage can be observed much sooner. This means that the tunnel system can be emptied in time before flooding occurs and major damage is caused. These observations can be done during the intervals between filling steps. Earlier experience has shown that during an interval of 10 to 20 hrs a steady state of leakage is achieved.

At Tjodan it was decided to fill the shaft in seven steps. During the intervals between filling periods, the level of water in the shaft was continuously and accurately monitored by an extra-sensitive manometer. By deducting the estimated natural groundwater leakage into the shaft and the measured leakage at the cone, it was possible to calculate net leakage into the rock.

The estimated average permeability coefficient of the rock masses from the results was 7×10^{-9} m/s, which can be considered very low.

No leakage into the powerhouse, the transport tunnels or the access tunnels was recorded. The powerhouse is located 100–150 m from the base of the shaft.

Total leakage from the shaft (water loss) was found to be about 1 1/s, including some 0.7– 0.8 1/s from the plug. (Leakage from the plug has subsequently diminished to 0.1 l/s.) This low figure was considered to be due to both the low permeability of the rock masses and the careful and thorough grouting in the shaft and plug area.

It can be mentioned that prior to the construction of Tjodan plant, at least seven pressure shafts/tunnels in Norway had been similarly filled in stages. Leakage from these averaged 0.5–5 1/s per km of tunnel, depending on the pressure of the water. At Tjodan the leakage was measured to approximately 1 1/s per. km.

COST SAVINGS OF THE UNLINED SHAFT

The design of Tjodan powerplant started out with the lower half of the pressure shaft being steel lined. This could have been done from a separate construction adit. In the detailed design phase it became evident that the rock mass conditions were favourable for a completely unlied shaft. An estimate was made which showed that not only would an unlined shaft be US $ 3

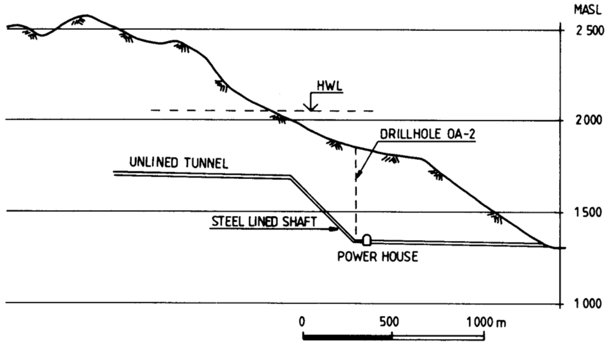

Fig. 1. Section along the tunnel system in the power station area.

million cheaper, but that it would probably be possible to shorten the construction period by at least two months. The saving resulting from bringing the plant on stream earlier was estimated at US $ 2.7 million adding up to a total av US $ 5.7 million.

The cost savings compared with a fully steel-lined shaft are subtantially higher.

References.

Palmstrøm, A. & Schanche, K. (1987) “Design features at Tjodan save time and money”. Water Power & Dam Construction, June 1987.

APPENDIX 3

DESIGN OF THE PRESSURE SHAFT AT THE ALFALFAL PROJECT IN CHILE

The Alfalfal Hydroelectric project which is presently under construction, is a run of the river plant with a planned installation of 160 MW. The owner is Chilectra Generacion. The scheme is utilizing the Colorado and the Olivares rivers. The static head is approximately 700 m, the power station beeing situated at about 1350 m.a.s.l. The project is located about 60 km east of Santiago.

The rocktypes encountered along the 25 km of tunnels consist of andesites, breceia, volcanic sediments, tuff and intrusive granite.

The pressure shaft is crossing subhorizontal strata of andesites, tuff and volcanic sediments.

A preliminary design of the pressure shaft was based on overburden criteria and FEM design charts, fig. 1. A load factor of 1.5 was used in order to ensure a safe margin against hydraulic jacking on joints and fractures. Several seepages in the area and also some small brooks were taken as indications on a generally high phreatic level.

In order to verify the assumptions the hole 0A-2, was drilled and tested, fig. 1.

Hydraulic fracture testing was done at 50 m intervals during drilling. The number of sucessfull tests were limited due to high permeability. Where tests succeeded they confirmed the FEM design charts.

The testing revealed strata with high permeability and also indications of perched groundwater tables.

Using the equation

$$Q = \frac{k \times 2\pi \times L \times \Delta P}{\ln D/r}$$

for a worst case estimate on leakage, the result was of the order of 1 m3/s based on permeability $k=5\times10^{-6}$ m/s and an average net head of 400 m.

This amount of leakage was considered unacceptable. The alterantives were steellining or grouting of the shaft.

Based on the probable high number of locations to be grouted, in practice a continuous grouting of the whole shaft, and an estimated grouting efficiency of 75%, it was revommended to use concrete embedded steel lining in the pressure shaft. The recommendation was accepted by the owner.

Effective sealing of drilled shafts in chalk

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SYNOPSIS

Because of their widespread distribution, the chalks of England and Europe are likely to see increasing use of drilled shafts either into or through them.

Seven drilled shafts have been constructed in chalk with three of them sealed successfully by initial grouting Four of the shafts were problems where remedial grouting caused serious economic consequences.

These shafts have been studied, along with drilled shaft sealing in general, to present the best possible grouting practice for insuring successful sealing with the initial grout placement.

INTRODUCTION

Because of its widespread occurrence in both England and Europe, shafts drilled into or through chalk probably will become common. The presence of aquifers within the total chalk interval and the unique characteristics of the chalk present both opportunities and problems of shaft sealing which need consideration by designers, contractors and owners if the maximum economic potential of drilled shafts is to be realized.

Seven shafts have been drilled in chalk at three sites. One site with one shaft is in the state of Alabama in the United States, one site with two shafts near Rouen, France, and four shafts near Killingholm, Humberside, England. Of the seven drilled shafts, the first three were successfully sealed by the initial grouting while the other four all suffered sealing problems with serious economic consequences. The quite variable results achieved in these seven shafts demonstrate the need to study and better understand what constitutes good grouting technique as applied specifically to chalk.

CHARACTERISTICS OF CHALK

Chalk is not a uniform material but varies, depending on the amount of foreign material included in it. White chalk characteristically is composed of two principal materials, molluscan debris and foraminifera which range in size from 10–100 microns within a matrix composed of coccoliths in the range of 0.5−4 microns in size. Commonly the smaller size will constitute from 75– 90% of the total chalk volume while the larger grains will constitute 10–25% of the chalk. The chalk will characteristically be very deficient in grains of greater than 100 microns and grains in the 4–10 micron range. The grains within the chalk usually are weakly cemented with calcareous material, primarily calcite. The degree of cementation can be variable itself, contributing to the variations which have been observed in the chalk. Hancock[1], Black[2], and Boswell[3] have described the charactristics of the chalk extensively.

Another characteristic of the chalk, described by Jenner and Burfitt[4], is the positive electrical charge carried on the individual chalk grains. Each grain then is surrounded by a halo of pearlized water.

It is the very fine grain size and the electropositive surface charge which the grains carry that makes chalk one of the most unforgiving materials for successful sealing of shaft liners.

The arbitrary and indefinite division usually applied between silt and colloidal size particles is 1 micron. Colloidal activity depends on the specific surface of the particle which varies with particle shape and on the surface potential. In some cases smaller size particles will not be colloidal and in others particles of greater than 1 micron and smaller size most assuredly will be colloidal in behavior and it is quite likely that the boundary between silt size and colloidal size particles will approach 4 microns, the upper limit of the coccolith fragments.

The weak cementing and fine particle size of the chalk presents problems in separating drilled solids from the circulating drilling mud during the drilling operation. Commution of the chalk is a requirement for drilling and additional commution can be expected from collisions of the drilled particles within the circulating system as they are circulated to the surface. Any drilled solids not removed from the circulating media on the surface will suffer further commution as they are recirculated in the drilling operation. It is not uncommon to have the drilled solids accumulate within the drilling mud until they constitute 30% or more by volume of the total solids content of the mud. Ideal drilling muds will maintain less than 10% total solids content as determined by recovery on a 200 mesh screen and in some instances efforts are made to maintain total solids as low as 5%. It is not the purpose of this paper to discuss the techniques for removal of drilled solids from mud systems but rather to point out that high concentrations of chalk solids can develop in the drill ing mud which will adversely affect the sealing of the shaft liner with grout.

CHARACTERISTICS OF DRILLING MUD

During the grouting phase of the shaft construction there is a complex interaction between the drilling mud, the Portland cement grout and the included drilled solids in the system. The characteristics of the mud will make an important contribution to the overall effectiveness of the grouting operation so it is appropriate to familiarize the reader with the basic objectives to be achieved with a mud.

Eight principal functions of a drilling mud are: 1) to remove cuttings from the bottom of the hole and carry them to the surface; 2) to cool the bit; 3) to plaster the wall of the hole with an impermeable cake; 4) to control subsurface pressures; 5) to hold cuttings in suspension when circulation is interrupted; 6) to release cuttings at the surface; 7) to support part of the weight of the drill pipe and casing in the hole; 8) to reduce to a minimum any adverse effects on the formation adjacent to the borehole.

The relative importance of any of these functions or combinations of functions is determined by the size of the hole being drilled, the formations being penetrated and the fluid content of those formations. Each of the functions of the mud and mud properties most influential on these functions are discussed individually and the ideal mud then is described.

Removal of cuttings

The removal of cuttings from the face of the bore is probably the most important function of the mud. Adequate cleaning of the face insures longer bit life and greater drilling efficiency. The mud rising from the bottom of the hole carries the cuttings to the surface. Under the influence of gravity, the cuttings are always falling relative to the motion of the mud but with adequate velocity, density and viscosity the net velocity of the cuttings will be ascending.

Cooling the bit

Heat is generated by friction on the bit where the teeth react with the formation being drilled. There is little chance for this heat to be conducted away through the formation so it must be removed by the circulating fluid. The heat having been transmitted from points of friction to the mud is lost to the atmosphere at the surface.

Hole wall

A good mud should deposit a tough filter cake on the borehole wall opposite formations with permeability to aid in consolidation of the formation and to retard the passage of fluid into or out of the formation. This property of the mud is improved by increasing the colloidal fraction of the mud with addition of bentonite and chemically treating the mud to improve the solids distribution.

Pressure control

Proper restraint of formation fluid pressures and support of the formation depends on the density of the mud. Under normal circumstances for shaft drilling, water as a circulating fluid or mud with some additional contained solids will be more than sufficient to balance formation pressure and prevent the entry of unwanted fluids into the borehole. Occasions of abnormal pressure which require specialized solids of higher specific gravity are rarely a factor in shaft drilling.

Cuttings suspension

Good drilling muds have properties that cause the solids being carried to the surface to be held in suspension due to a gellation or thixotropy which develops when circulation is interrupted. On resumption of circulation, the mud will revert to its fluid condition and carry the suspended particles to the surface.

Cuttings release at the surface

A mud with high suspending ability for removal of cuttings from the borehole will be one which only reluctantly releases the cuttings at the surface. Either long surface retention time or mechanical solids removal equipment is required to remove cuttings effectively from the suspending mud.

Support

With increasing hole size and depth, the weight supported by surface equipment becomes increasingly important. Since both drill pipe and the casing or lining material to be installed are buoyed by a force equal to the weight of mud displaced, an increase in mud density will result in a reduction of the total weight which the surface equipment must support.

Support of adjacent formation

When the borehole penetrates a formation of high in situ stress, the formation may spall because of the asymmetry of the in situ horizontal stress field. Some formations are reactive with water and can hydrate and slough when exposed to the drilling mud. Also when drilling highly fractured formations, the formation may tend to slough because of sliding along the naturally occurring or induced fracture planes which is promoted by the lubrication of any penetrating liquid from the drilling mud.

Increasing the density of the drilling mud will work to stabilize the borehole wall and minimize, if not prevent, spalling or sloughing as a result of the in situ stress or natural fracturing within the formation. Formation reaction with the drilling mud can be controlled by the appropriate chemical treatment of the mud prior to the penetration of a reactive formation.

The ideal mud is one which has sufficient density to offer support to the borehole yet reverts to very low density when flowing beneath the drill bit to effectively scour cuttings from the bottom of the hole. This mud will also have a relatively high viscosity when carrying cuttings to the surface along with adequate gel strength to hold these cuttings in suspension when circulation is interrupted. When this hypothetical mud reaches the surface it will revert to low density, low viscosity and low gel strength so that the cuttings will drop rapidly from the mud. It is obviously impossible to achieve this hypothetical ideal mud, so a series of compromises must be made to get the desirable properties for a most nearly optimum mud.

Drilling muds almost universally contain bentonite as the gelling material to impart the desirable rheological properties to the mud and this is usually the most satisfactory and economical material. In some instances, water alone can be used for the circulating fluid but frequently this is not the case.

Muds containing bentonite are characteristically intolerant of contamination by the calcium ion. Because the reaction of Portland cement to water liberates a large number of free calcium ions, the drilling mud is normally quite intolerant of cement contamination unless special protective measures are taken[5–7].

REACTIONS OF PORTLAND CEMENT

The metamorphosis of Portland cement and water from a slurry of anhydrous particles to a set grout is a complex series of chemical reactions. During reaction time a number of intermediate compounds are formed which create a liquor rich in these intermediates. As the reaction proceeds, the intermediate compounds further react as does the water itself so that the finished mater- ial is a hydrous crystalline mass with almost no free water and considerable strength. For our consideration the intermediate compound of greatest importance is calcium hydroxide which supersaturates the cement liquor. Part of the calcium hydroxide will crystallize as free calcium hydroxide subject to leaching but the majority will react with other intermediates to become a part of the strength-creating series of compounds. During the time while the cement liquor is rich in calcium hydroxide, it can react with surrounding materials as well as with the cementacious materials. This is the critical period during which unwanted reactions can occur.

Nearly all bentonite drilling mud formulations are intolerant of high concentrations of calcium ions. This is especially true when the pH of the mud is increased by the addition of hydroxil ions. The effect on the mud of increasing the calcium ion concentration is to flocculate it with a very rapid increase in its apparent viscosity, yield point and gel strength. This action tends to form a layer of flocculated mud at the mud/grout interface, promoting viscous fingering or channeling of grout through the mud and leaving pockets or channels of mud beneath the advancing front of grout.

The flocculation of the bentonite in the mud is undesirable but with the usual concentration of 5% or less by volume, it may not by itself present an unmanageable problem. The greatest problem is probably the loss of the power to suspend drilled solids included in the mud.

The calcium ion, having a positive charge, tends to cause a collapse of the diffused double layer around the bentonite particles where gravitational attraction then brings them together to form flocs.

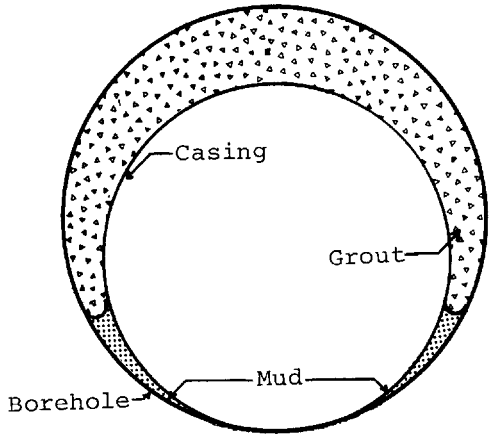

Fig.1 Effect of casing borehole contact

Just as the calcium ion causes flocculation of the bentonite in a drilling mud, the hydroxil ion in the cement liquor will neutralize the diffuse double layer around the positively charged chalk particle and aggregates of chalk grains will begin to form in the flocs of increasing mass. Further increases in the anionic concentration will cause the individual flocs to band together in an extensive gel structure.

Robinson[9], along with Witten and Cates[10], describe the process of viscous fingering of one liquid through another.

Effective shaft sealing then becomes a matter of preventing or minimizing the adverse effects of the contact between the grout slurry and the drilling mud. Good grouting practices have been established through the years by painful trial and error. The basic good practice does not vary regardless of the formations to be sealed, but because of the unforgiving nature of chalk the observance of good grouting practice is even more imperative than when other formations are to be sealed.

THE EFFECT OF THE CASING

The objective of grouting a casing or a liner into a borehole is to support the casing and seal the annulus so that there will be essentially no movement of unwanted fluids either up or down the annulus. To achieve these goals requires that the in situ annular fluid be replaced by the grout material.

To achieve nearly perfect replacement of in situ fluid by grout material begins with the design of the casing itself. Provision for grouting must be made and provision for maintaining an adequate standoff of the casing from the borehole wall must also be provided. The displacement of the in situ fluid by the grout material is analogous to the tremi placement of concrete where the difference in density causes the in situ fluid to essentially float on top of the displacing grout. Since both the mud and chalk are thixotropic, if there is continuity of either left in the annular space, it will provide a channel for the migration of fluids within the annulus. If a water-bearing zone is crossed by a stringer or channel of mud or chalk solids it can be displaced by the formation water to provide a leakage channel with potentially undesirable consequences.

If the casing or liner is in contact with the wall of the borehole there will be a dead area on either side of the contact where the in situ material cannot be displaced by the ascending grout. Fig. 1 is a plan section through a borehole which diagramatically illustrates trapping in the vicinity of a casing borehole contact point. Even eccentricity of the casing within the borehole will cause a non-uniform rise of grout with the potential for trapping channels of undesir able material within the body of the grout. Vaughn[11], studied the effect of the flow of non-Newtonian fluids in eccentric annuli, primarily directed toward the grouting of oil and gas well casing where the clearance between the borehole and casing is much smaller than it is in a shaft. Even though the eccentricity is less in shaft grouting than oil and gas well grouting, the effect is still present and it is illustrated in Fig. 2.

To prevent actual borehole contact and/or unacceptable degrees of eccentricity of casing within the borehole, standoff or centralizing devices are installed on the casing prior to its installation. These devices normally are a T section with several of them arranged circumferentially around the casing with the long axis parallel to the longitudinal axis of the casing. Fig. 3 illustrates a ring of centralizers on a section of casing. The centralizers are not normally extended to establish full contact with the borehole wall but rather extend from the casing wall enough to assure a minimum standoff and minimize the eccentricity of the casing in the borehole.

Because some eccentricity of the casing in the borehole is inevitable, to minimize the effects of the eccentricity, as illustrated in Fig. 3, multiple grout lines are commonly used to assure maximum circumferential continuity of the grout around the casing. The number of grout lines used varies with the diameter of the casing and should never be less than four. In the larger diameter casings it is common to use six or eight grout lines equally spaced circumferentially around the casing.

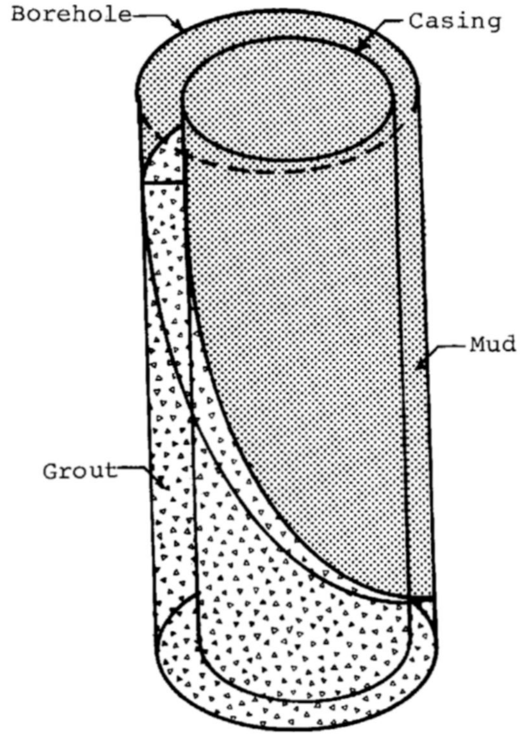

Fig. 2 Effect of casing eccentricity

To assure the placement of grout lines during the grouting operation, grout line guides are installed on the casing. These guides usually are pipe with a sufficient diameter to accept the grout lines. On occasion, square tubing has been used for grout line guides but most commonly it is standard pipe. These guides function only to provide guidance for the grout lines so they are penetrated by a series of holes through which the grout can flow during the grouting operation. These openings usually are flame cut with the area of each hole being equal to or greater than the cross sectional flow area of the grout line. The penetrations of the grout line guides are commonly in a spiral pattern around the guide with spacing between holes usually about 2 ft. (600 mm). Fig. 4 shows the placement of grout line guides around a section or joint of casing.

In two of the four shafts which required remedial work, continuous grout line guides were used. Only short sections of pipe attached to the casing of approximately 3 m vertical intervals were used on the other two. Unfortunately, the grout lines either failed to thread through all of the guides or the guides caused the lines to snag. As a result, several intervals of grout line were pulled apart and dropped in the annulus. This probably contributed to an uneven grout rise and the trapping of mud and chalk solids in the annulus.

MUD DISPLACEMENT BY GROUT

As previously discussed, the contact between a chalk-solid-laden bentonitic mud and Portland cement grout has the potential for serious consequences, making placement of the grout critical. One procedure which has been used successfully in three shafts grouted through chalk was to replace the drilling mud with water at the conclusion of the drilling. When this can be done, possible interactions between grout and mud or chalk solids can be avoided completely. The replacement of the mud with water is not always acceptable because the support of the borehole is reduced by the lower density of water compared to mud. Additionally, if any formations have been penetrated which are reactive with fresh water, the replacement of the mud may result in undesirable hydration of the sensitive formation with consequential sloughing and poor grouting results.

For the balance of this discussion we will consider that for some reason, either real or perceived, the replacement of the drilling mud with water is unacceptable.

Parker, Ladd, Ross and Wahl[12] des-cribe the techniques of grouting or cementing at low displacement rates. They found that the effectiveness of displacement is proportional to the density and gel strength of the displaced and displacing fluids. If the grout density exceeds the density of the mud by as much as 5 lbs/U.S.gal., then the gel strengths can be essentially equal. If the grout and mud are of the same density (an unlikely circumstance) then the gel strength of the grout must exceed that of the mud by at least 25 lbs/100 sq.ft. (a difficult contrast to obtain). In the determination of the required density in gel strength contrast, careful consideration should be given to the possible eccentricity of the casing in the borehole. If there is reason to

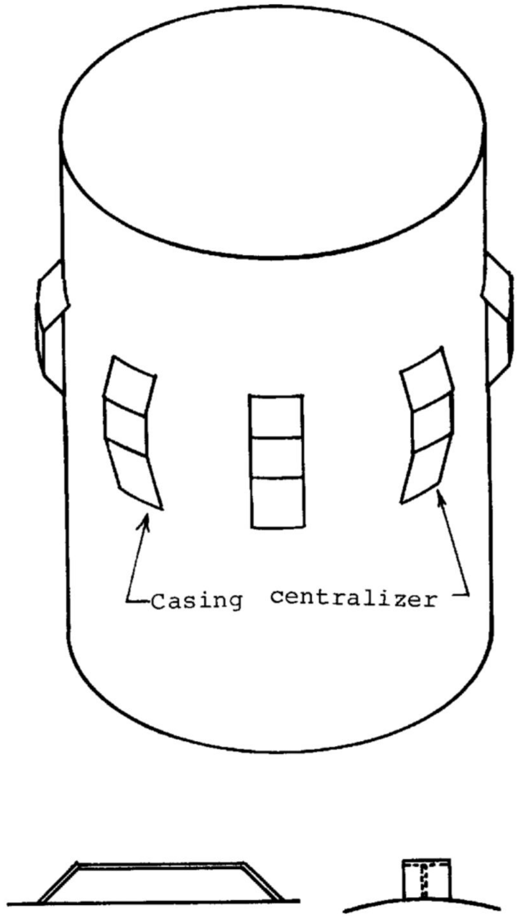

Fig. 3 Typical centralizing or stand-off system

expect considerable eccentricity (the borehole survey indicates crookedness) the density and gel strength contrast should be increased.

The adverse effects of the contact between mud and Portland cement grout can be minimized or eliminated by the maintenance of an appropriate buffer zone of treated water and treated drilling mud between the ascending grout and the in situ mud[13]. The use of an appropriate dispersant in the drilling mud will reduce the gel strength of the mud and minimize the effects of contact between mud and grout. These dispersants normally are introduced into the mud system just prior to grouting and then followed by a quantity of water in which the dispersant is mixed to form a buffer between the ascending grout and the drilling mud. The selection of a dispersant is normally controlled by the particular nature of the. mud and common dispersants include sodium acid phyro-phosphate (SAPP), calcium lignosulfonate, ferrochrome lignosulfonate, tannins and proprietary dispersants available through grouting contractors.

In the four problem shafts, no dispersant-treated water was used as a buffer nor was any significant volume of water pumped prior to grout placement.

FIELD PROCEDURE

The final step in the perfecting of the annular seal with grout is the installation of casing and grouting operation. This is the last opportunity to enhance the grouting effectiveness and possibly the most important portion of the total grouting planning and operation.

When the drilling of the shaft is completed, if the replacement of the drilling mud with water is acceptable, this is the ideal time for the replacement. For the purpose of this discussion we will assume that replacement of mud by water is not acceptable.

Prior to withdrawing the drilling tools from the shaft, the mud should be circulated and conditioned by treating to remove as many of the drilled solids as economically possible. This could consist of a complete replacement of the mud in the shaft with freshly blended mud which contains no drilled solids. Because of hole volume, the total replacement of drilling mud may be prohibitively expensive and for this reason we will assume that it is not done.

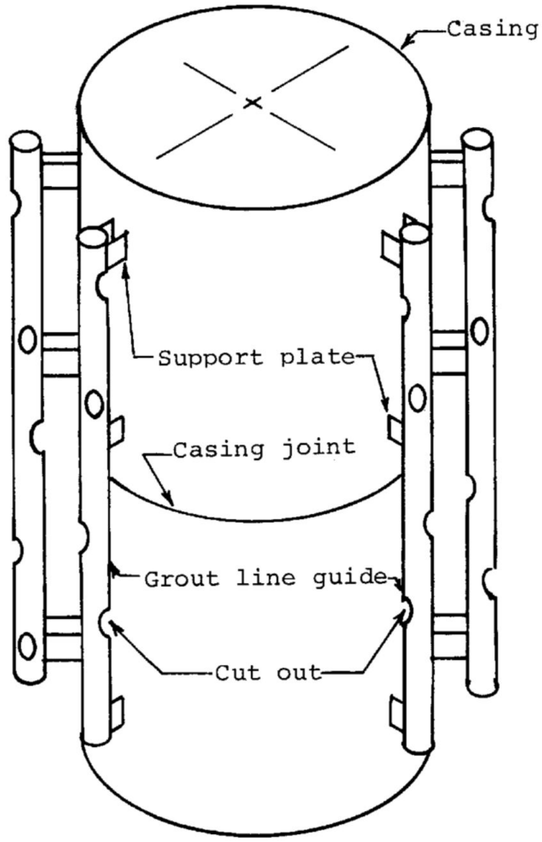

Fig. 4 Grout Line quides

The circulation and conditioning of the mud should, as a minimum, constitute the circulation of one complete hole volume. Ideally the circulation and conditioning of the mud will continue until the measured properties of the mud returning from the shaft are identical with those of the mud being pumped into the shaft.

In the four shafts where remedial work was necessary, almost no conditioning of the mud was done prior to installing casing and the volume of drilled chalk solids in the mud was at an unacceptably high level.

After conditioning the mud, drilling tools are withdrawn and the casing installed. Under the most usual circumstances, the casing is sealed and the buoyance of the fluid in the shaft will float a portion of the weight of the casing, minimizing the load on surface equipment. There will be a point where the casing will be completely buoyant and it must be sunk into the shaft by the addition of water or mud inside the casing. Casing installation continues until all of the casing is in the shaft.

After the casing has been installed and suspended at the surface, the grout lines are installed in the grout line guides and circulation established through each of the lines individually. After the initial circulation through each line has demonstrated that each line is open, circulation continues by pumping through all lines. During this circulation the properties of the mud returning from the hole will be observed and reconditioning of the mud will be performed if necessary to obtain the minimum weight, gel strength, plastic viscosity and calcium content consistent with hole stability. Even if the mud properties are uniform during this period of circulation, a minimum of one hole volume of mud should be circulated. The initial circulation of mud through each individual grout line can determine if some mishap has resulted in the plugging of an individual line. If this has happened, then the grout line should be removed from the hole and cleared. On reinstallation of that grout line, the pumping procedure should resume and when it is determined that all grout lines are clear, pumping should be through all the lines.

After the mud in the annulus has been conditioned, a volume of water containing a dispersant or mud thinner should be injected through the grout lines to further reduce the gel strength and viscosity of the mud in the annulus. The selection of dispersant or thinner is based on the type of mud that has been used in the drilling of the hole. The volume of treated water usually should not be less than the volume calculated to fill 30 m of the annulus.

After the treated water mud flush has been injected into the annulus, an equal volume of water should follow, giving an effective buffer of approximately 60 m of hole before any grout is injected. On completion of the water injection, grouting should begin with grout being injected uniformly into all grout lines. The first stage of grout will usually be short to provide an anchor at the bottom of the casing, eliminating the risk of floating the casing up the hole. After the first stage of grout has been pumped, the grout lines should be moved up the hole to assure that each of the lines is clear of the first stage of the grout and then each line flushed individually with water to clear any grout from the line. At the option of the contractor or owner, the grout lines may be completely removed from the shaft at the completion of a grout stage.

After sufficient time has elapsed for the first stage of grout to achieve a strong initial set (usually considered to be about 300 psi compressive strength) preparations for the second stage of grouting can begin.

For the second stage of grout, the grout lines will be lowered in the shaft and the top of the first stage of grout determined by tagging with the grout lines. The depth to the top of the first stage of grout is measured by each line and recorded. The depths to the top of grout should be uniform with less than a 1 m spread between the highest and lowest level of grout in the annulus. The grout levels measured should be compared to the calculated rise of grout based on the volume injected in the preceding stage. If the rise of grout was significantly less than that calculated, the calculations should be checked and, assuming they are proven correct, it probably indicates that a zone of lost circulation has thiefed some portion of the grout, or that the hole is much enlarged. If the grout level is higher than the level calculated from the first stage of grout, it is probable that there has been channeling of grout or caving of the borehole wall in the annulus. No further grouting should take place until the condition is evaluated and the most probable cause determined, then if corrective action is possible, it should be taken.

Assuming that circumstances were normal on tagging the top of the first stage of grout, preparations can begin for subsequent grouting. As with the first procedure, water is injected which has been treated with the same thinner or dispersant as in the preceding stage, with the volume calculated sufficient to fill about 30 m of the annulus followed with a volume of untreated water, again equal to about 30 m rise in the annulus. At the beginning of the conditioning, a small volume of treated water should be injected through each grout line to insure that the line is clear. If one or more lines is plugged, the offending line should be withdrawn from the shaft and cleared prior to any further injection. After the treated water and clear water has been injected into the annulus, the injection of grout simultaneously through each grout line can begin.

The second stage of grout normally will be of greater volume than the first stage, and if it is desired the grout lines can be withdrawn during grouting.

The procedure for grout line withdrawal during grouting is that when the calculated rise of grout in the annulus has exceeded the length of one joint of grout line, injection into one line will cease until one joint has been withdrawn from the hole, then injection will begin again through that line. This procedure is followed, withdrawing one joint of pipe from each string, until one joint has been withdrawn from all the strings, at which time injection through all the strings will continue until the calculated rise of grout again exceeds the length of one joint of grout pipe when the withdrawal procedure will be repeated.

As with the first stage of grouting, all grout lines shall be removed completely from the shaft or to a level well above the calculated top of the grout and individually flushed with water.

The procedure described for the second stage of grout will be followed until the annulus has been filled with grout. If the final stage of grout is to be less than 60 m then all of the mud remaining in the annulus should be flushed out of the hole with treated water and clear water before the final stage of grout is cast. Fig. 5 illustrates the grouting procedure.

If there is a wide variation in grout levels at the conclusion of a stage, remedial action should be planned, based on the findings. One solution might be to wash the top of the grout extensively through the grout lines and then place a short stage of grout through the lowest line or lines. This stage should be of slightly greater volume than that calculated to even the grout surface. This short stage should be allowed to set and then operations resumed. If the surface is now level, then normal grouting can resume. If the surface is not level, then another short filler stage may be appropriate.

If, after casting a stage of grout, the grout level is found to be much higher than anticipated, there is little that can be done at this stage of the operation beyond an attempt to wash grout and possible caved material from the annulus by circulating mud through the grout lines in an attempt to wash any sloughed material from the annulus. The possibility of this being effective is remote, but it should be attempted. This particular region of the shaft may be a candidate for remedial work from inside the shaft after grouting has been completed. This possibility is unlikely if care has been used in drilling to assure hole stability and further care exercised in the placement of each grout stage.

In the case where mud was required to protect an isolated zone or zones until they were covered by grout, the grouting procedure can be modified. When the highest problem zone has been covered by grout, the mud remaining in the annulus can be displaced with water before following stages of grout are placed.

It is impossible to overemphasize the necessity for adequate pretreatment of the hole before grouting. It is much easier to condition the mud with drilling tools in the hole than after casing and grout lines are in the hole. For this reason the mud should be conditioned to optimum properties prior to withdrawal of the drilling tools. If this is accomplished, then the initial conditioning of the mud after the casing and grout lines have been installed will be minimal and the primary emphasis will be on the establishment of a buffer between the ascending grout and the mud.

The preceding discussion offers general procedures for grouting casing in a shaft and does not eliminate the need to analyze in detail the specific conditions in a borehole to plan a program specifically for that borehole.

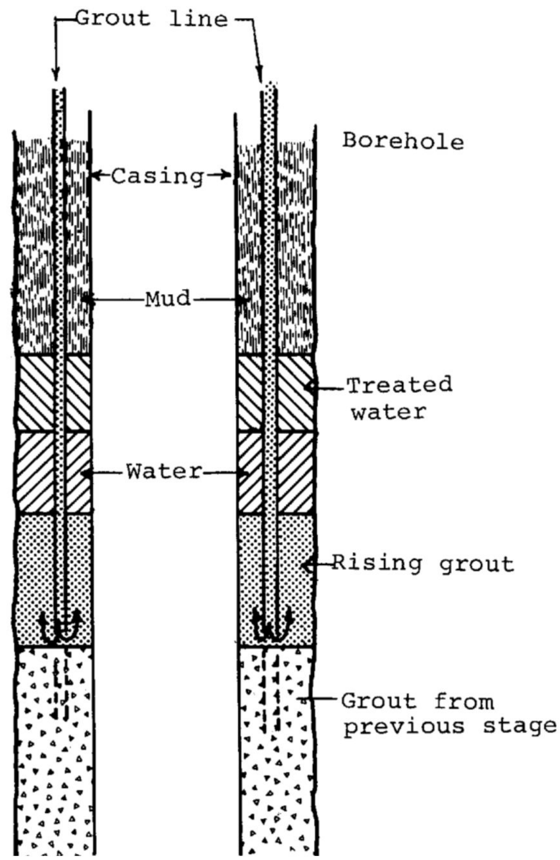

Fig. 5 Casing grouting

References

1. Hancock, J.M. "The Petrology of the Chalk", Proc. Geologists' Association of London, vol. 86, part 4, 1975, p. 499–535.
2. Black, Maurice. "The Constitution of the Chalk", Proceedings of the Geological Society of London, no. 1499, session 1952–53, 29 May 1953. 1xxxi– 1xxxvi.
3. Boswell, P.G.H., Muddy Sediments: Some Geotechnical Studies for Geologists, Engineers And Soil Scientists, W.Heffer & Sons, 1961. p. 40–51, 96–98.
4. Jenner & Burfitt, J.B., "Chalk — An Engineering Material", Catalogue No. 8159 Institution of Civil Engineers, London 1976, p. 39.
5. Gray, George R. & Darley, H.C.H., Composition and Properties of Oil Well Drilling Fluids, 4th ed., Gulf Publishing 1980, p. 138–173.
6. ——, Drilling Fluid Engineering Manual, Dresser-Magcobar, 1977.
7. Rogers, W.F., Composition and Proj erties of Oil Well Drilling Fluids, 3rd ed. Gulf Publishing 1963. p. 448.
8. ——, Cementing Technology, Dowell-Schlumberger, 1984, p. 5–6.
9. Robinson, A.L., "When Are Viscous Fingers Stable?", Science vol. 228, 17 May 1985, p. 834–836 ——, "Fractal Fingers in Viscous Fluids," Science vol. 228, 21 May 1985, p. 1077–1079.
10. Witten, T.A. & Cates, M.E., "Tenuous Structures from Disorderly Growth Processes", Science vol. 232, 27 June 1986. p. 1607–1612.
11. Vaughn, R.D., "Axial Laminar Flow of Non-Newtonian Fluids in Narrow Eccentric Annuli", SPE Trans, vol. 234, 1965. p. II 277–80.
12. Parker, P.M., Ladd, B.J., Ross, W.M., and Wahl, W.W., "An Evaluation of a Primary Cementing Technique Using Low Displacement Rates", SPE Paper 1234, Denver, CO, 1965.
13. —— , Dowell Cementing Manual, Dowell Division of Dow Chemical Co. 1981.

Construction of shafts for the Greater Cairo Wastewater Project

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The Greater Cairo Wastewater Project has involved to date the construction of 14 km of tunnel and 40 shafts at depths up to 25 m below ground level in water bearing alluvial sands and silts. Future works will necessitate further shaft and tunnel construction. All the shafts, except one, are circular and range in size from 4.85 m to 10.0 m in finished internal diameter. One was constructed as a triple cell caisson as a contractor's option to facilitate launching of the tunnel boring machines. The shafts serve to provide connections between the trunk sewer tunnel and the existing sewerage system, and also future connections to further sewer tunnel works. All shafts were sunk under compressed air at pressures up to 2.2 bar.

The ground conditions are described. Shaft sinking methods used on site are compared with the Engineer's design and typical sinking records are given.

INTRODUCTION

The Greater Cairo Wastewater Project is a major public health engineering scheme to expand, renew and rehabilitate the existing Cairo sewerage system.

One of the principle elements of this scheme has been the construction of a spine tunnel and associated shafts on the east bank of the city. The purpose of this paper is to describe the design and construction of the east bank shafts.

A total of 40 have been constructed and more are planned under future development of the scheme. Construction of two very large shafts at Ameria pumping station do not form a part of this paper.

The various works, construction sites and other features described are shown on the accompanying figures.

THE PROJECT

Prior to 1914 Cairo had no sewerage system. In that year the first stage of a plan prepared by C Carkeet James was implemented.

Over the years the initial system of the east bank has been extended and a scheme to serve Giza and other developing areas on the west bank of the Nile was commissioned.

The present scheme provides for expansion of the system to cope with demand into the 21st century, the main elements of which are shown on Figure 1.

Present construction and planned future works comprise 43 km of tunnel in water bearing alluvial sands and silt, 4 km of rock tunnel, 10 major pumping stations, 33 km of twin box culvert, 43 km of collectors and sewers and 2 treatment works. In addition to this new work, there has been a significant programme of rehabilitation of the existing system.

EAST BANK SHAFT WORKS

The shafts constructed under the present east bank scheme are situated on the main spine trunk sewer tunnel and several branch tunnels. The former is of 4 m and 5 m finished internal diameter whilst the latter range from 1.2 m to 2.8 m finished internal diameter.

The downstream end of the shaft and tunnel works is situated in the north eastern suburbs of the city at Ameria, Much of the route lies beneath Port Said Street which follows the line of an old canal and lies approximately parallel to the Nile in a north south direction. The upstream end of the present works lies in the south of the city at Ein el Sierra on the edge of Fostat. A future extension of the scheme is planned beneath the limestone hills of Fostat to Maadi.

All the shafts, except one, are circular, and range in size from 4.85m to 10.0 m in finished internal diameter. One was constructed as a triple cell caisson as a contractor's option to facilitate launching of the tunnel boring machines.

Construction was carried out in three contracts, all being undertaken simultaneously. Contract 3 covered the northern, or downstream end of the route, Contract 4 the central portion and Contract 12 the southern, or upstream, part of the route.

SOIL AND GROUNDWATER CONDITIONS

Cairo lies on river deposits at the apex of the Nile delta and is bound on the east by the limestone cliffs of Gabal Mokattam and on the west by the Pyramids Plateau. Both the meandering of the river and man's occupation of the area for the last 5000 years have had a great influence on the variations to be found in the deposits beneath Cairo.

Pre-construction site investigations showed that the soils within the trunk sewer area generally consisted of alluvium deposited by the Nile. This was overlain by made ground (fill).

The made ground was up to 13m thick. It comprised typically miscellaneous rubbish in a silty clay matrix but both depth and composition were complicated by the presence of abandoned and infilled canals, lakes and ponds which contained rubbish, refuse and construction waste.

The typical stratigraphy of the alluvial soils consisted of:

1. Silty clay and clayey silt.
2. Interbedded clay, silt and sand.
3. Sand and gravel.

The silty clay and clayey silt layer generally consisted of soft to very stiff grey or brown, silty clay to clayey silt and occasional sand and silt partings. It was found in thicknesses up to 11 m but in many areas it was not encountered due to the depth of the fill material.

The second soil unit was a transitional stratum between the silty clay above and the sands below. Though not always present this interbedded stratum of clay, silt and sand was found to be quite variable and in thicknesses of up to 10 m. A very silty fine sand was usually encountered at the base of these interbedded materials.

Beneath the clays, silts and fine sands lay a very thick (believed to be greater than 50 m) stratum of sand. This comprised typically a silty fine sand at the top grading to coarse sand with some gravel with depth. Gravel layers also occurred.

Standard penetration tests carried out in this stratum showed an increase in density with depth ranging from very loose to medium dense at the top of the layer to dense to very dense at the bottom of the deepest boreholes.

The site investigations indicated that typically the shafts on Contracts 3 and 4 would be constructed in granular material over the lower half of their depth. However, in the southern part of the east bank shaft works on Contract 12, very much more variable ground conditions could be anticipated. Much greater thicknesses of the silty clays, clayey silts and interbedded materials could be expected with the sands only occurring towards the base of the shafts. One shaft on Contract 12 was to be constructed wholly in limestone, it being at the edge of the Fostat hills.

Figure 2 shows the ground conditions along the route of the main spine trunk sewer tunnel.

The groundwater in the trunk sewer area, and throughout Cairo, is affected by a variety of factors including:

- the Nile
- seepage from Mokattam escarpment
- irrigation
- leaking water supply and wastewater pipes
- water abstraction from wells
- construction activities including dewatering
- and low annual rainfall

Leakage of water supply and wastewater pipes within the city are believed to have a significant affect on the static water table causing perched water tables within the clay stratum.

Groundwater levels in the north eastern part of the trunk sewer area were artifically depressed by abstraction at the Cairo North wellfield. Typically the groundwater was at a depth of 4 m in this vicinity. Moving away from the influence of the wellfield the groundwater assumed a depth of 1 to 2 m below ground level.

ENGINEER'S SHAFT DESIGN

Shafts were required for access to the tunnels, as vortex drop shafts, and as connecting shafts to the existing sewerage system.

The design of the shafts took into account both permanent works and construction requirements.

It was decided that access shafts on the main spine trunk sewer tunnel should be not more than 1 km apart. Where changes of alignment or branch tunnel connections occurred this spacing was reduced.

It was anticipated from the outset that the shafts would be sunk under compressed air. They were to be constructed through water bearing made ground and alluvial soils in very congested streets and often close to existing buildings. It was necessary to ensure that settlement of adjacent structures was kept to a minimum, that construction could proceed as quickly as possible and would not be slowed by excessive friction in the granular soils and that air losses particularly at shallow depths could be contained at tolerable levels.

All shafts were designed as circular reinforced cast in-situ concrete structures to approximately 10m below ground level, 2. 5 m above the tunnel eyes, with the remainder of the shaft being p.c.c. segmental rings terminating in a substantial mass concrete plug at the base.

These shafts were designed so that the cast in-situ section could be sunk as a pneumatic caisson with a cutting edge bolted to p.c.c. segments cast into the shaft underside. The shafts were to be fully secured in the ground by grouting up of the choker annulus and the cutting edge removed.

Thereafter, the shafts were to be sunk to final depth by building on the p.c.c. segmental lining using traditional underpinning methods and compressed air. After reaching final depth the segmental rings received a secondary in-situ concrete lining.

It was felt that this two stage method would minimise both the risk of settlement adjacent to the shafts and the likelihood of caissons “hanging up” due to excessive friction in the sands. Caisson sinking through the made ground would also help to reduce air losses.

CONTRACTOR’S ALTERNATIVE DESIGN

The contractor for Contracts 3 and 12 decided to submit an alternative design for shaft construction. He also revised the concept of Shaft 3–a main tunnel shield launching shaft, by the addition of two circular outer pods, to the 10 m dia. circular shaft initially required.

These were temporary works structures designed to facilitate the launching of 2 no. shields, heading in opposite directions, one to Ameria (downstream) and the other to Shaft 5 (upstream) at Souk el Samak.

The main design alterations which were proposed by the contractor included:

a) Insitu r.c. shafts for full depth with integral toe and cutting edge, with increased wall thickness compared to the original design.
b) Corresponding increase in vertical and circumferential reinforcement to withstand possible greater pressures, torsional and tension loads during shaft sinking.
c) Reduction in thickness of the base plug due to the increase in weight of the structures.
d) Openings for future tunnel connections formed of mass concrete with addition of radiused eye trimming bars to compensate for this.
e) Elimination of the secondary insitu concrete lining and the p.c.c. rings.
f) Temporary composite steel and r.c. air decks fixed to the shafts.

Shaft 3 was a special case due to the three pod configuration which required a complete re-design. The main central shaft remained as 10.0 m i.d. The outer pods intersected this and were also 10.0 m i.d.

At the intersection, the common walls were designed with 6.5 m dia. circular openings facing approx north and south to permit introduction and passage of the 6.12 m external dia. main tunnel shields.

This arrangement required very heavy 32 dia. H.Y. reinforcement “columns” at the node points and around the openings.

The contractor’s alternative proposals were accepted in principle by Ambric in April 1985 and effectively made the design and execution of the shaft construction the complete responsibility of the contractor, with the proviso that the system was demonstrated successfully on two non-critical, but typical, shafts prior to complete acceptance of the system.

The first two shaft designs were completed, checked and approved by early June 1985 and construction commenced in the same month with the two trial shafts being successfully sunk and plugged by mid August.

CONSTRUCTION EXPERIENCE

Contract 3 and Contract 4 commenced at approximately the same time, whilst Contract 12 started some time later.

The contractor for Contract 3, having proposed an alternative scheme for shaft construction, was required to demonstrate that the method would be satisfactory. Two smaller dia. shafts were selected for this demonstration.

Both of these shafts were on a branch line tunnel and were chosen as they represented typical but varying ground conditions and were also on the most advanced sites at this early stage of Contract 3. Shaft 5A5V1 was not at all critical, being a vortex shaft at the end of the branch, but Shaft 5A2 was crucial to the launch of the branch line tunnel shield.

Initial excavation was carried out to the ground water level. An outer bentonite reservoir/guide wall of approximately 600 mm thick mass concrete and 300 mm clear of the caisson toe all round was then constructed. The base of the excavation was blinded with limestone aggregate and sand which was graded to be as uniform in level as possible.

A set of radially placed 100mm×50mm timbers was then set to precise level on this surface and embedded with sand to secure them in position. The prefabricated segments of the steel cutting edge were then set to correct position, levelled up on these timbers and welded together.

The shutters were prefabricated steel segments rolled to the inner and outer radii required and bolted together through abutting flanges. The fixing to the structure was achieved by over ties and large diameter pins which located into recesses formed in the previous lift.

The shutters were erected with timber spacers between the flanges of the outer segments to form the choker ring which increased the diameter of the bottom 2 m high pour by 100 mm. The inner shutter was fitted to accommodate a 45 degree splay from the cutting edge. The reinforcement was then fixed and the bottom lift concreted. Good curing was essential in the hot climate and the shutters were insulated with polystyrene and the concrete covered after pouring and kept wet and shaded. Figure 3 illustrates the initial construction stages. Figure 4 shows the steel cutting edge, sand bed and timbers, reinforcement for the splay and the inner shutter.

For the remaining lifts, the shutters were raised using a special spreader frame and all spacers were removed to reduce to standard diameter. The shafts were concreted up to the air deck level for which box outs were formed. Figure 5 shows the guide wall, reinforcement to the tunnel eye and the inner shutter.

The shaft launching was achieved by placing a horizontal cruciform of splayed end 300 mm×300 mm timber blocks and wedges beneath the concrete splay of the cutting edge section.

These were eased away from the concrete splay by the controlled removal of the wedges and blocks to ensure that the shaft remained vertical. The levelled bearing timbers were manually withdrawn during this process but in some instances snapped off.

The exterior annulus was then filled with bentonite after the cutting edge had sufficiently penetrated the ground.

The caisson was sunk in free air until water ingress at the base prevented further safe progress. The air deck was constructed of a framework of heavy steel beams embedded into the caisson walls and infilled with reinforced concrete. Standard Gowring air locks and trunking were bolted onto the air deck and supported within a steel framework

With the caisson pressurised to balance the invert water pressure at the current level, the sinking process recommenced. The internal splay was underdug, as shown in Figure 6, using manual methods in these initial smaller shafts until the shaft moved downwards.

Monitoring using a water level within the shaft and reference marks against set datums, at four locations at right angles to one another, on the outer concrete face ensured that the necessary corrections during excavation were made to maintain the shaft plumb. These stages are illustrated on Figures 7 and 8.

Concurrent with the sinking operation, the upper wall lifts were added to the final level.

As the depth of sinking increased, the excavated wet sand was discharged directly onto the top of the air deck to act as kentledge for the remainder of the sinking operation. Bentonite was added to the reservoir to maintain the desired head as required and the air pressure gradually increased to counter the build up of water pressure.

When the cutting edge achieved the desired level, mining ceased and the air pressure was increased by 0.1 to 0.25 bar to prevent the caisson sinking further. The bentonite annulus was then displaced by cement grout to lock the caisson within the ground and the concrete plug poured. Figure 9 shows the final stages of shaft construction.

General Procedures Developed

The ability to sink full depth caissons was considered as having been successfully proven by these two demonstrations and the proposal by the contractor for the remaining 14 no. caissons on Contract 3 was agreed subject to satisfactory design. The method having been accepted for Contract 3 was also adopted for Contract 12.

Although the ground conditions on Contract 12 were much more variable than those on Contracts 3 and 4, including much interbedded silts, sands and clays, the caisson sinking methods employed on Contract 3 proved equally successful on Contract 12.

Shaft No 3 was critical to the programme for the main tunnels and staged approval of the design was given to avoid any delay. Thus, this large three pod caisson was already under construction when the demonstration of the technique was being concluded.

The technique was the same in principle, but on a much larger scale which required that the two outer cells be concreted simultaneously and the centre cell independently, thus maintaining even loading for sinking purposes. As the concrete mass was greater, thermocouples were attached to the reinforcement to monitor concrete temperature. In the event, the maximum specified of 70 degrees C was not exceeded.

The shaft was sunk through the silty clay strata in free air with concrete lifts added to the pods as sinking progressed. Excavation was undertaken simultaneously in each pod using converted Smalley excavators plus hand trimming.

The air deck trunking and air locks and compressed air were installed when about 2m of silty clay remained and the caisson sunk to final level under compressed air. Figure 10 shows the caisson partially complete with the air locks installed, concreting formwork in place and reinforcement being fixed.

The geometry of this three pod caisson made it sensitive to equal balance on all pods during sinking and during the latter stages it became reluctant to sink without overcutting.

As this could have caused a blowout with inevitable inrush of ground and loss of bentonite, the air pressure was lowered slightly for a very brief period to reduce uplift. This technique was successful and the caisson was successfully sunk to level without mishap.

This technique was used on two other of the early caissons to assist in the final sinking. The problem here appeared to be due to:

a) a tendency to overpressurise the caisson to achieve dry working conditions and to prevent excessive sinking
b) a reluctance to add the full calculated kentledge, particularly at an early stage of the sinking process

The results of these practices were that a) produced a greater uplift on the caisson than was allowed for at a given depth and reduced the overall downward weight. In addition, air on occasion escaped beneath the cutting edge and resulted in “boiling” of the bentonite reservoir. This inevitably disturbed the vertical wall of sand above the choker ring and caused minor collapses which increased friction on the walls and hence resistance to movement.

The effect of b) tended to make it drop in sudden jumps which also caused disturbance to the side walls of the bentonite reservoirs.

In addition to this, it was found necessary to remove virtually all the sand from the splay area just above the cutting edge.

The cause of these problems became apparent as experience was gained of the techniques required in the prevailing conditions and the air pressures were more carefully controlled and the kentledge of wet sand kept to a maximum at all times. Upon adoption of these techniques, there were no further cases of blowing of the bentonite well and the caissons sank much more gradually in a controlled manner to very accurate levels and locations.

The concreting of the base plug caused many problems in the early stages, particularly in the larger diameter caissons due to the relatively large volumes required e.g. Shaft 3 required approx 300 cu.m. and Shaft 4 in excess of 100 cu.m.

Until satisfactory routes for pipes and non-return valves had been evolved, the technique of pumping concrete was totally unsuccessful.

The alternative of placing by 1 cu.m. capacity skips via the air lock and trunking was very slow indeed—a maximum rate of 8 to 9 cu.m. per hour being possible at the best. Thus, the plugs were cast in two or three stages with vertical dowel bars placed between lifts. After resolution of the concrete pumping problems, this technique was abandoned and the concrete placed by pump with a flexible end hose at up to 20 to 25 cu.m. per hour.

Further amendment to the techniques initiated by the contractor, particularly on the smaller diameter shafts, were to excavate and concrete the bentonite reservoir wall to a maximum depth corresponding to the water table and to concrete the walls to full height prior to commencement of sinking under compressed air.

This improved the rate of shaft construction dramatically, the sinking process taking only 8 to 10 days to sink 20 metres. An additional bonus of this method was improved directional stability during sinking due to the greater depth of bentonite reservoir concrete wall which effectively acted as a guide wall.

Typical caisson sinking records are shown on Figures 11 to 13. Each figure shows the soil conditions through which each caisson was sunk, the increase in air pressure as the cutting edge sank deeper and the kentledge used.

Engineer’s Design

The contractor for Contract 4 elected to undertake shaft construction in accordance with the Engineer’s design. The sequence of construction was as follows.

Excavation was carried out to just above the groundwater table, the ring beam built and a steel cutting edge erected on a concrete pad. A choker ring and one standard ring were bolted to the cutting edge, reinforcement and formwork fixed and concrete poured for the first lift of the caisson.

The concrete pad was then broken out from beneath the cutting edge and the whole unit lowered by 1.5 m. An exterior jacking system was then fitted to control verticality and the rate of sinking. Concreting of the caisson was completed including a compressed air deck corbel. Figure 14 shows the caisson for Shaft 9 under construction with the exterior jacking system in operation.

Bentonite was introduced into the anular space between the ring beam and the caisson by injecting it through grout pipes cast into the caisson and choker ring. The air deck and compressed air equipment were installed and air pressure applied. The contractor for Contract 4 chose a T lock air system with a large muck lock which permitted the use of much larger skips than is possible with a Gowring lock.

Excavation was undertaken inside the caisson using a small back acter, muck was removed in skips through the air lock and the caisson was simultaneously lowered on the jacking system by 750 mm. The jacks were then moved, one by one, to fit the next jacking position and caisson excavation and sinking continued.

The excavation process is shown in Figure 15.

As the caisson reached the design level of 2.5 m above tunnel eye, a timber cribbing landing pad was installed onto which the caisson was lowered. The annulus was grouted up with neat cement grout injected through the grout holes in the choker ring thus displacing the bentonite upwards. The cutting edge was exposed and removed, a circular lifting beam installed and construction continued by underpinning. The air pressure was increased to keep the water just below the ring of segments being installed and kentledge was correspondingly increased.

Each ring of segments was grouted as soon as it was completed, every third ring was back grouted. At final depth, the shaft was completed by pouring an unreinforced concrete plug.

Every effort was made to minimise the air losses by keeping the cutting edge buried and the air pressure sufficient only to ensure that rings could be built in the dry. In addition, despite the use of a hydrotite gasket, losses occurred through joints. This was due to the air pressure preventing water coming into contact with the hydrophilic gasket. Once air pressure was removed, the gasket expanded and conventional caulking was not needed.

Nevertheless, losses of air through the ground were in some cases substantial, especially when excavating through coarser material. This caused the groundwater table in the vicinity of the shafts to rise, threatening adjacent properties. To reduce this threat, some shafts close to buildings were provided with a 2 m thick anulus of chemically consolidated ground extending from the underside of the clay/silt layer to 1 m below plug level.

The 8 m–10 m finished diameter shafts were of 23 m average depth. The durations of caisson sinking varied fromseven to fifteen days, underpinning took from eighteen to thirty eight days. Maximum air pressure was 2.2 bar and maximum consumption 280 cu.m/min, lowest maximum consumption on any shaft was 75 cu.m/min.

The 6 m finished diameter shafts were of 21 m average depth. The durations of caisson sinking varied from seven to eleven days, underpinning took from ten to sixteen days. Maximum air pressure was 1.9 bar and maximum consumption 165 cu.m/min, lowest maximum consumption was 40 cu.m/min.

COMMENTS AND CONCLUSIONS

The dense saturated sands were a good medium in which to employ the pneumatic caisson technique in conjunction with bentonite lubrication for shaft construction.

The caissons were generally sunk to their design depth within the specified tolerance of +/−50 mm. On Contract 3, the two demonstration shafts exceeded their design depth by 150 to 200 mm due to inexperience in the final stages of sinking. Verticality was maintained within the 1 in 100 specified but a shape problem developed in the upper lifts of one or two shafts due to inattention to detail when setting the shutters.

The use of steel shutters resulted in low grout loss and a very smooth clean concrete surface.

Construction of the shafts occurred within 5m of some very poorly constructed local buildings but no settlement or damage was observed due to these operations. Escape of compressed air did cause basement flooding of nearby buildings in some cases. The problem seemed to be worse on Contract 4 than on Contracts 3 and 12.

Close control of sinking pressures helped to minimise this problem and indeed on Contracts 3 and 12 air losses were generally reduced to low amounts.

The results of grouting of the bentonite annulus was observed at tunnel eye breakouts and confirmed virtual 100% replacement of the bentonite by grout.

It was interesting to note, however, that the 50 mm wide annulus formed by the choker ring had reduced to approximately 10 mm at the tunnel horizon level.

Concrete quality was generally good. However, during training of local labour on the early stages of the project, some compaction was not as good as it might have been.

There were one or two occasions when the compressors failed for a short time but fortunately the air loss from the caissons was slight and no harm resulted.

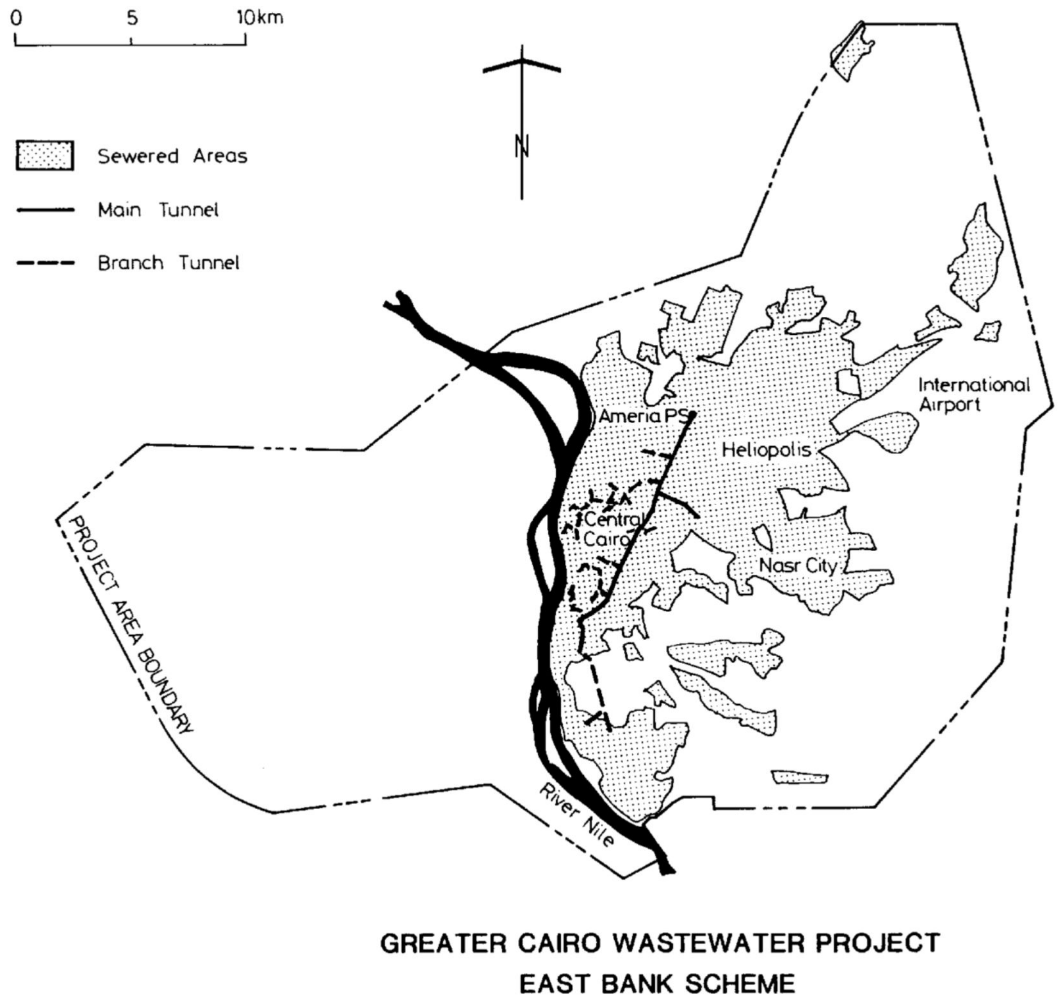


Figure 1

Conclusions

The adoption of the Engineer’s scheme on Contract 4 proved to be a sound construction method resulting in shafts which adequately met the Specification requirements. There were no major problems during sinking and no damage done to third parties or adjacent properties. The configuration of the shafts and tunnels on this contract together with the programming of the work meant that there was not the same urgency for completion of shaft sinking as there was on Contract 3.

The decision by the contractor for Contracts 3 and 12 to propose and undertake an alternative method to suit his construction approach proved successful. The proposal for the multicell solution to Shaft No 3 was a very imaginative well thought out one to solve the cramped conditions for launching two shields.

ACKNOWLEDGEMENTS

This paper is published with the kind permission of the Chairman of the Cairo Wastewater Organisation and the Board of Control of American British Consultants.

Figures 3, 7, 8 and 9 are based on method statements prepared by Cairo Wastewater Consortium.

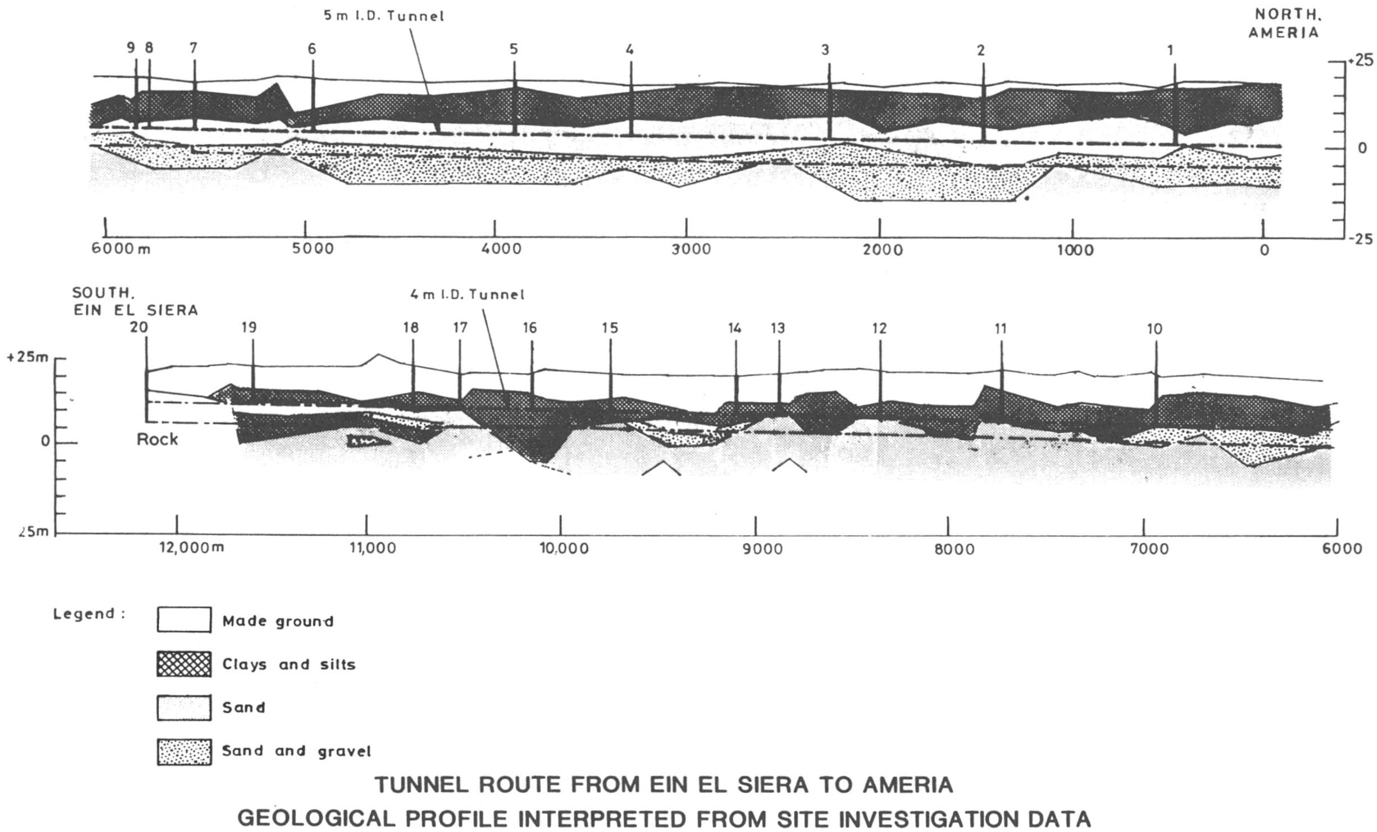
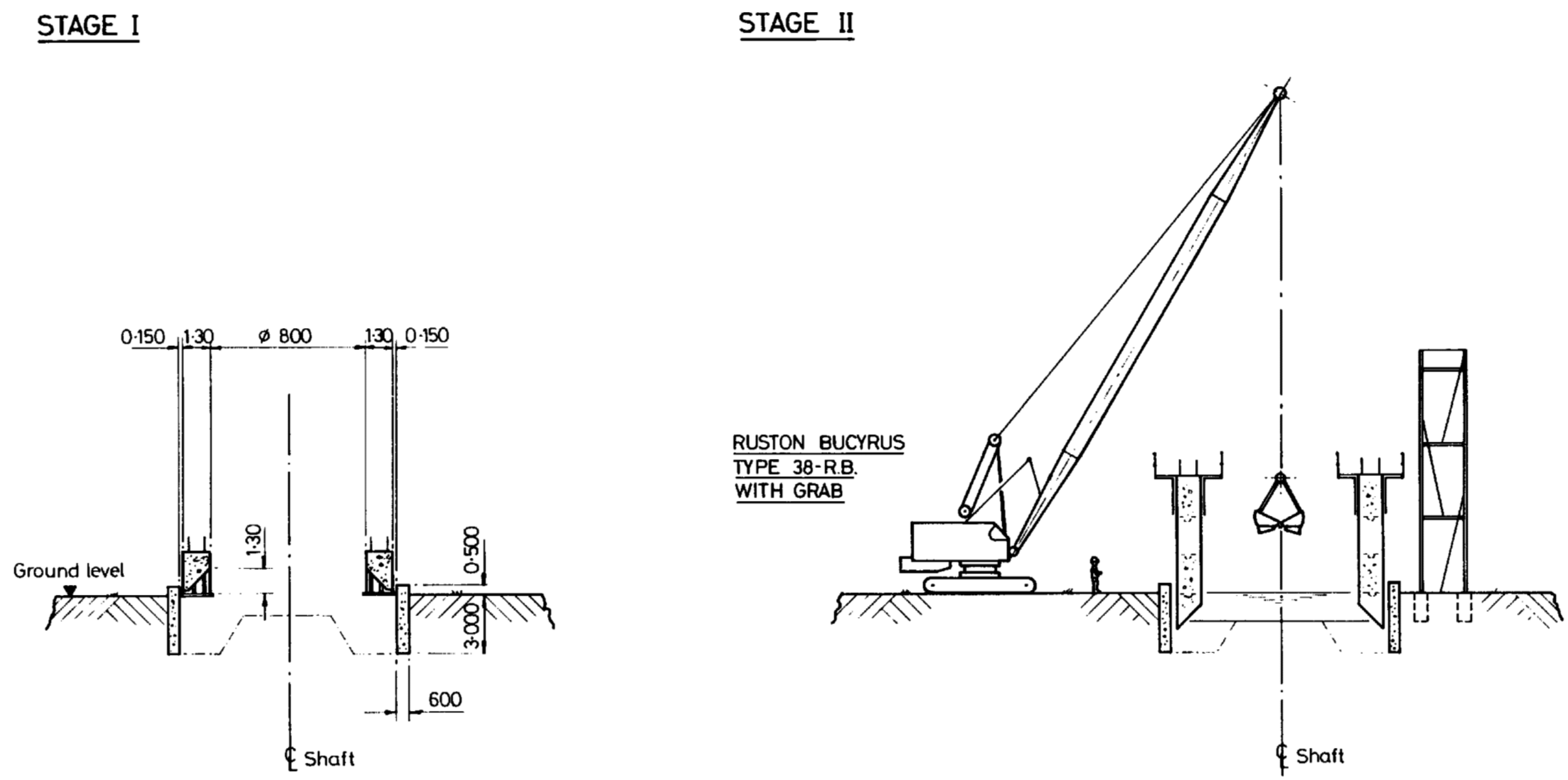
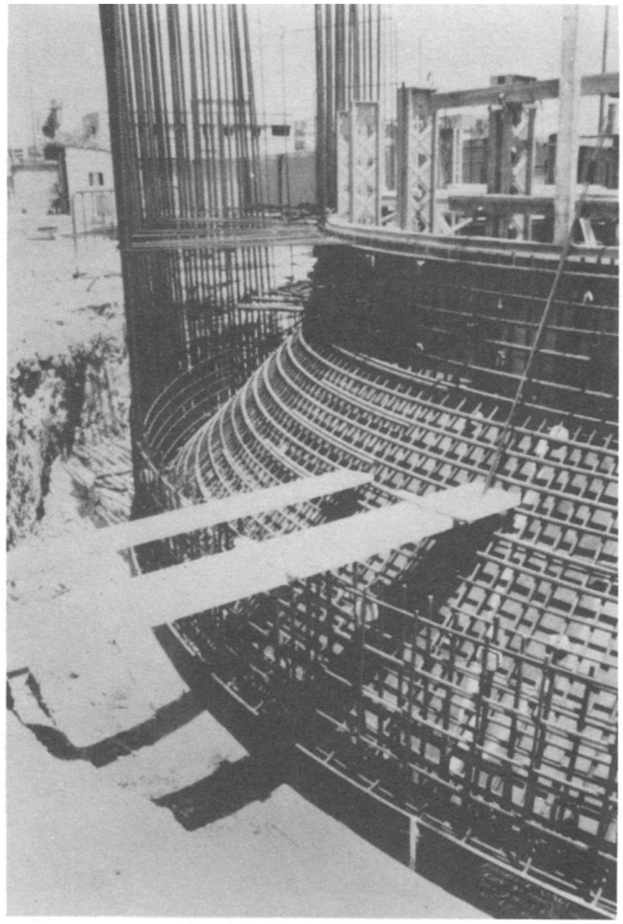

5 m I.D. Tunnel
NORTH.
AMERIA
9 8 7 6 5 4 3 2 1
+25
0
-25
6000 m
5000
4000
3000
2000
1000
0
SOUTH.
EIN EL SIERA
4 m I.D. Tunnel
20 19 18 17 16 15 14 13 12 11 10
+25m
0
Rock
25m
12,000m
11,000
10,000
9000
8000
7000
6000
Legend :
Made ground
Clays and silts
Sand
Sand and gravel
TUNNEL ROUTE FROM EIN EL SIERA TO AMERIA
GEOLOGICAL PROFILE INTERPRETED FROM SITE INVESTIGATION DATA

Figure 2



CONTRACTS 3 AND 12
CAISSON SHAFT CONSTRUCTION

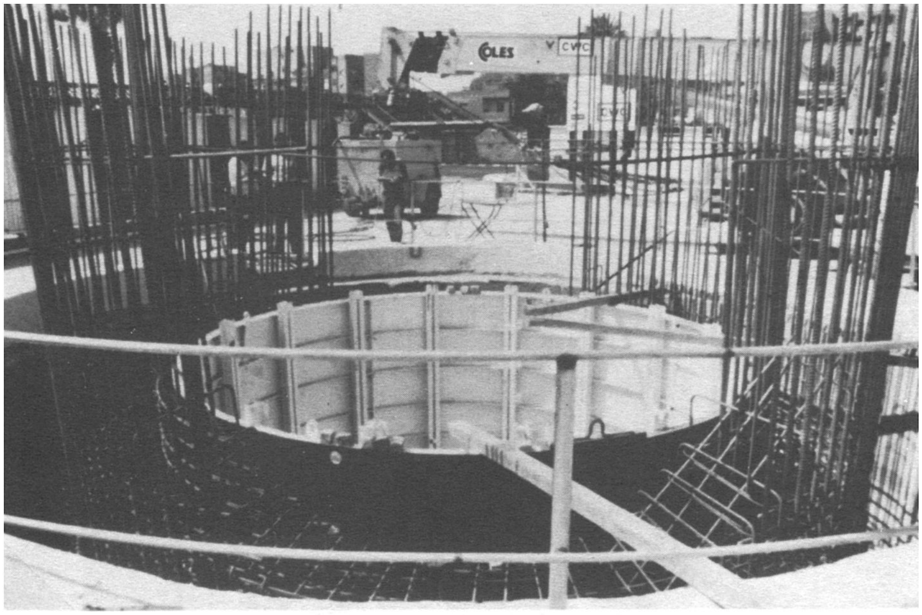
Figure 3



CONTRACTS 3 AND 12

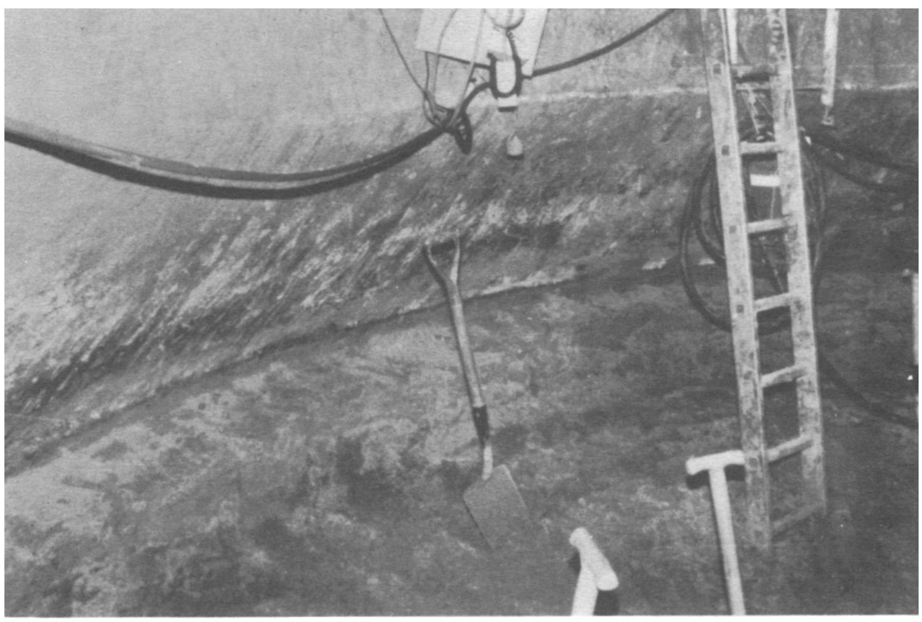
EARLY STAGE OF CAISSON CONSTRUCTION SHOWING STEEL CUTTING EDGE, SAND BED, SPLAY REINFORCEMENT AND INNER SHUTTER

Figure 4



CONTRACTS 3 AND 12

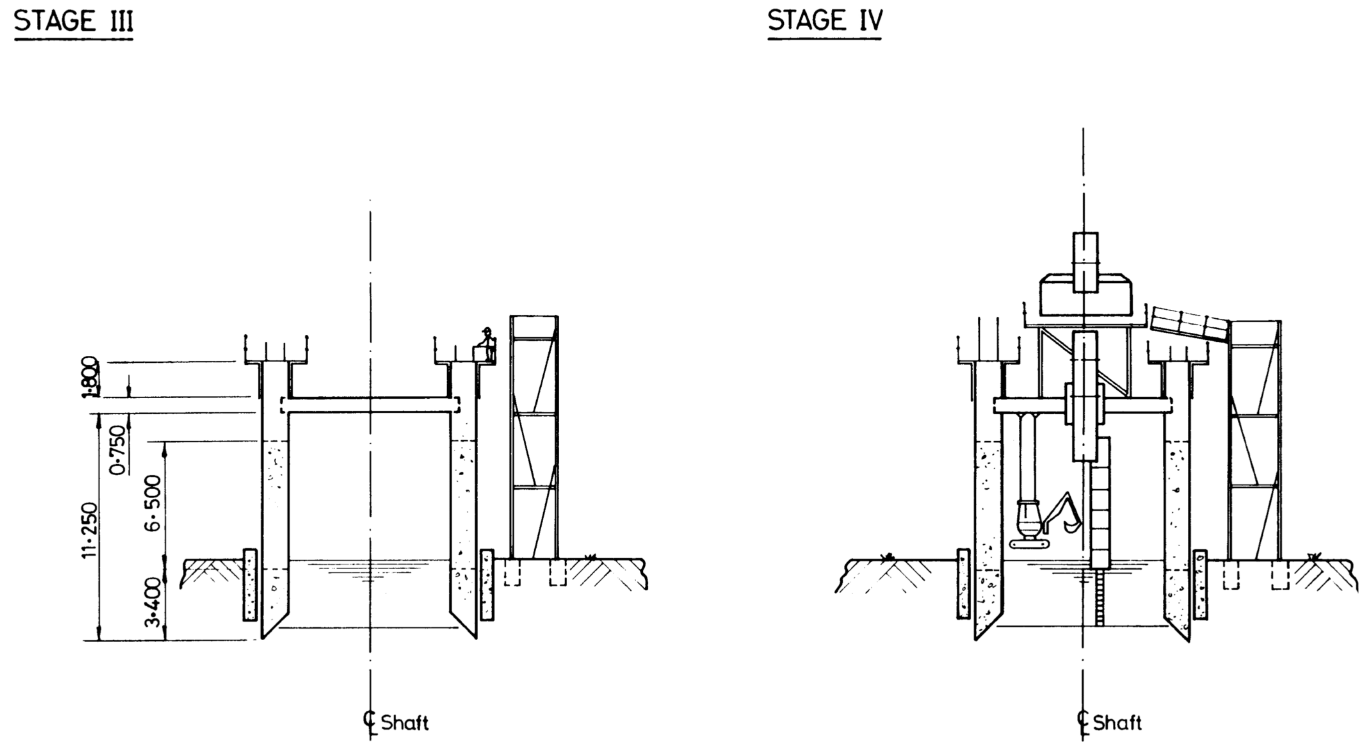
CAISSON GUIDE WALL, TUNNEL EYE REINFORCEMENT AND INNER SHUTTER



CONTRACTS 3 AND 12

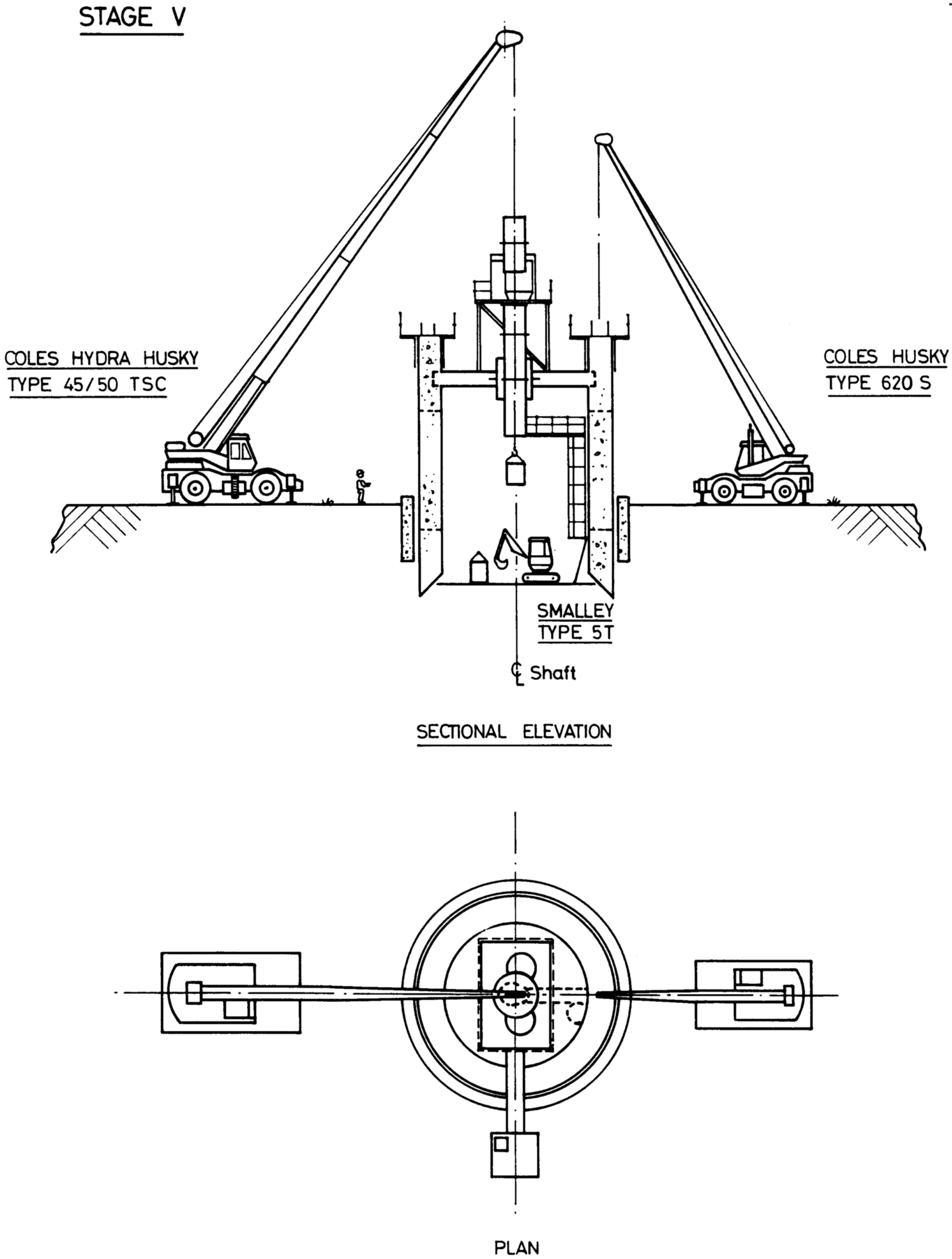
EXCAVATION INSIDE CAISSON ADJACENT TO CUTTING EDGE

Figures 5&6



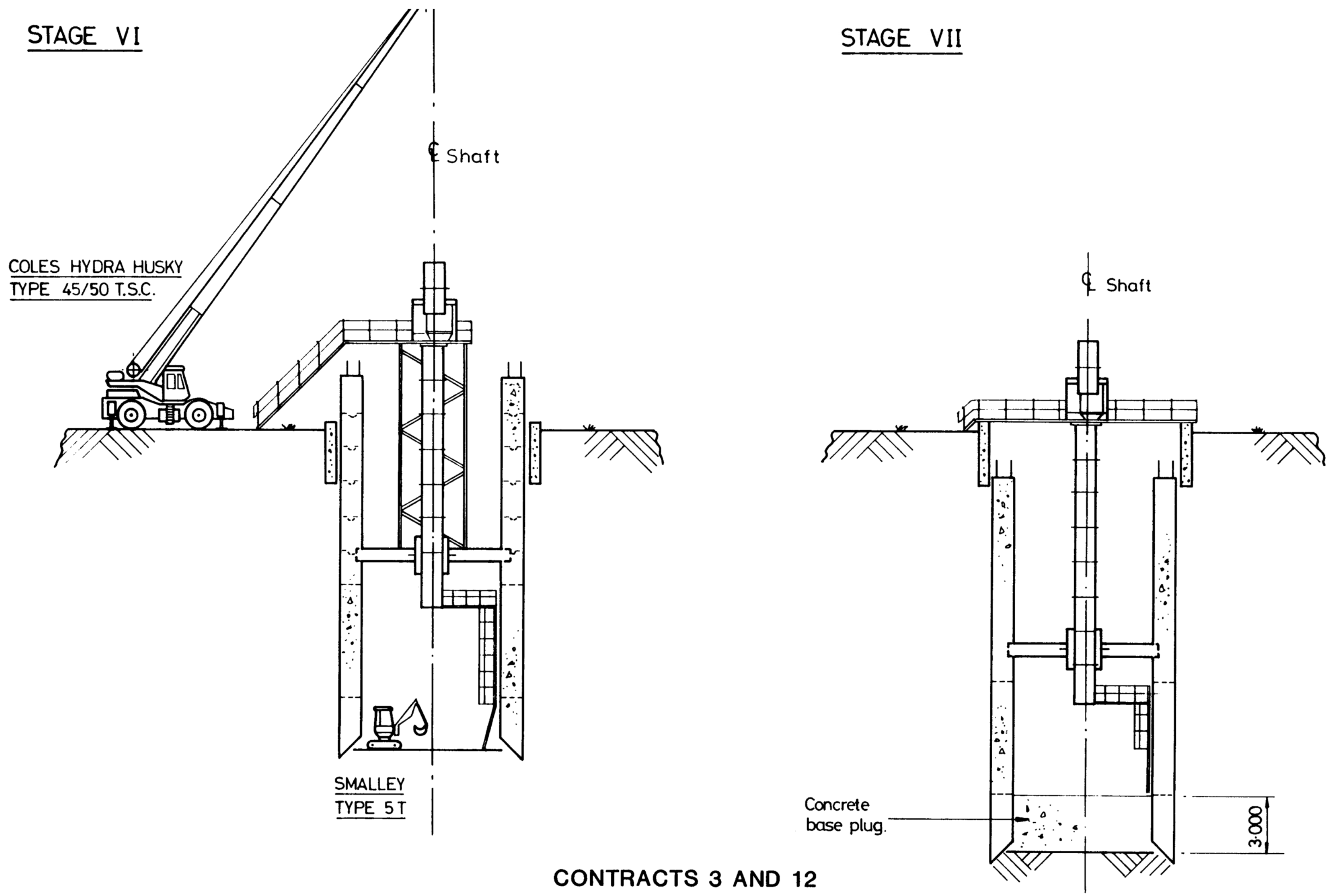
CONTRACTS 3 AND 12
CAISSON SHAFT CONSTRUCTION

Figure 7



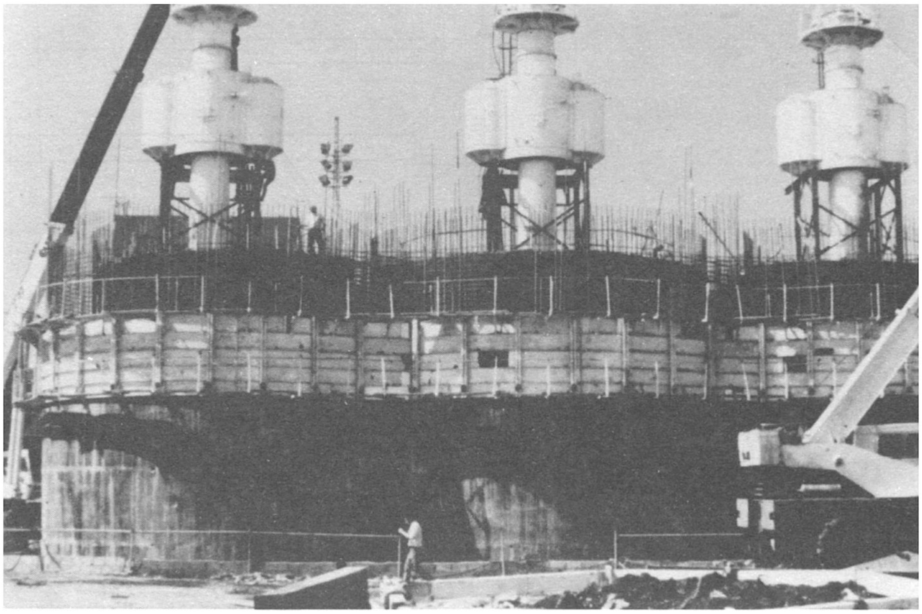
CONTRACTS 3 AND 12
CAISSON SHAFT CONSTRUCTION

Figure 8



CONTRACTS 3 AND 12
CAISSON SHAFT CONSTRUCTION

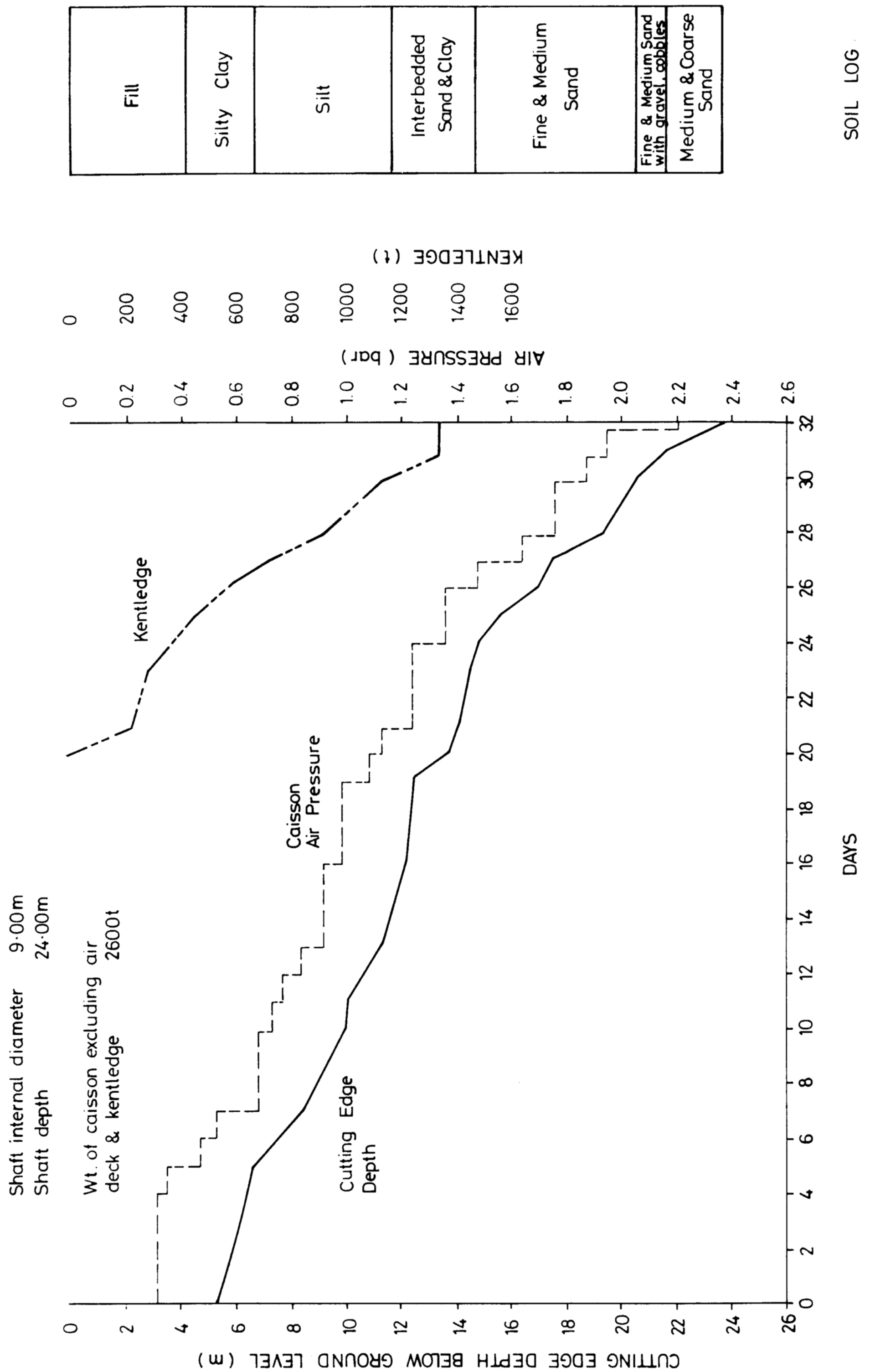
Figure 9



CONTRACT 3

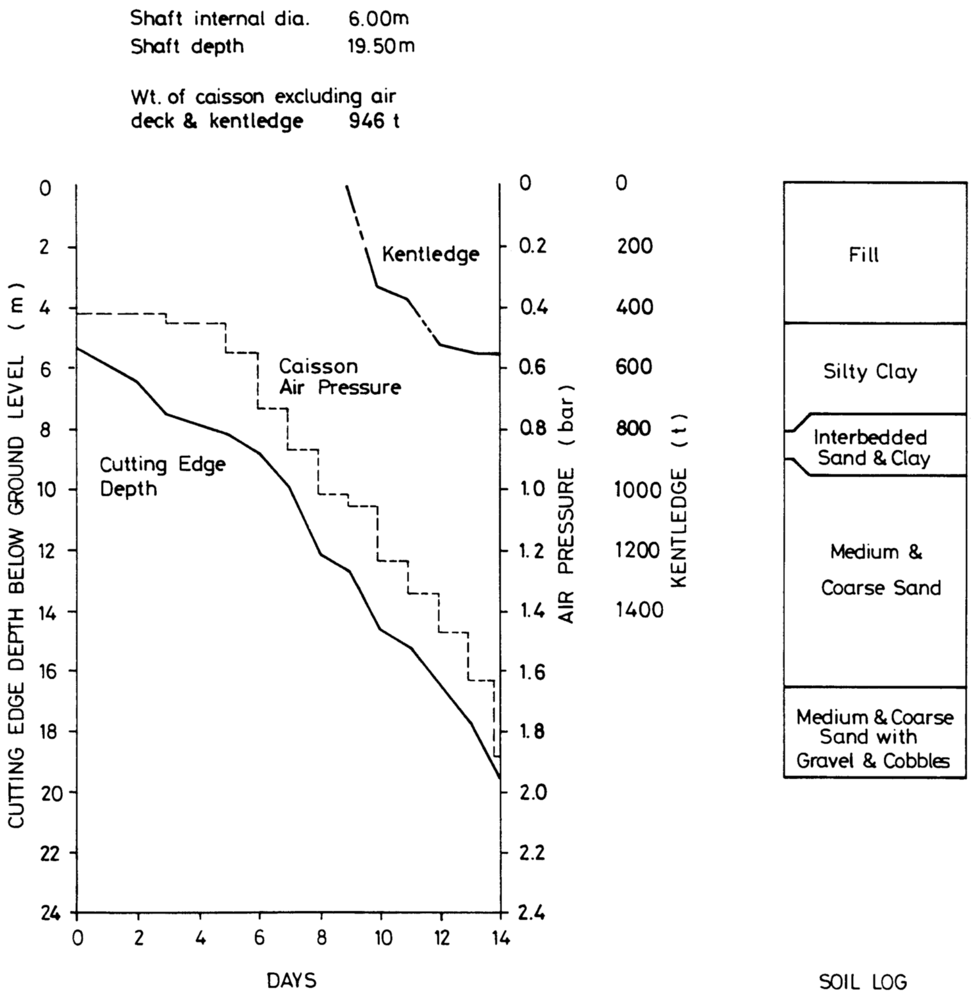
SHAFT 3 TRIPLE POD CAISSON WITH AIR LOCKS INSTALLED, CONCRETING FORMWORK IN PLACE AND REINFORCEMENT BEING PLACED

Figure 10



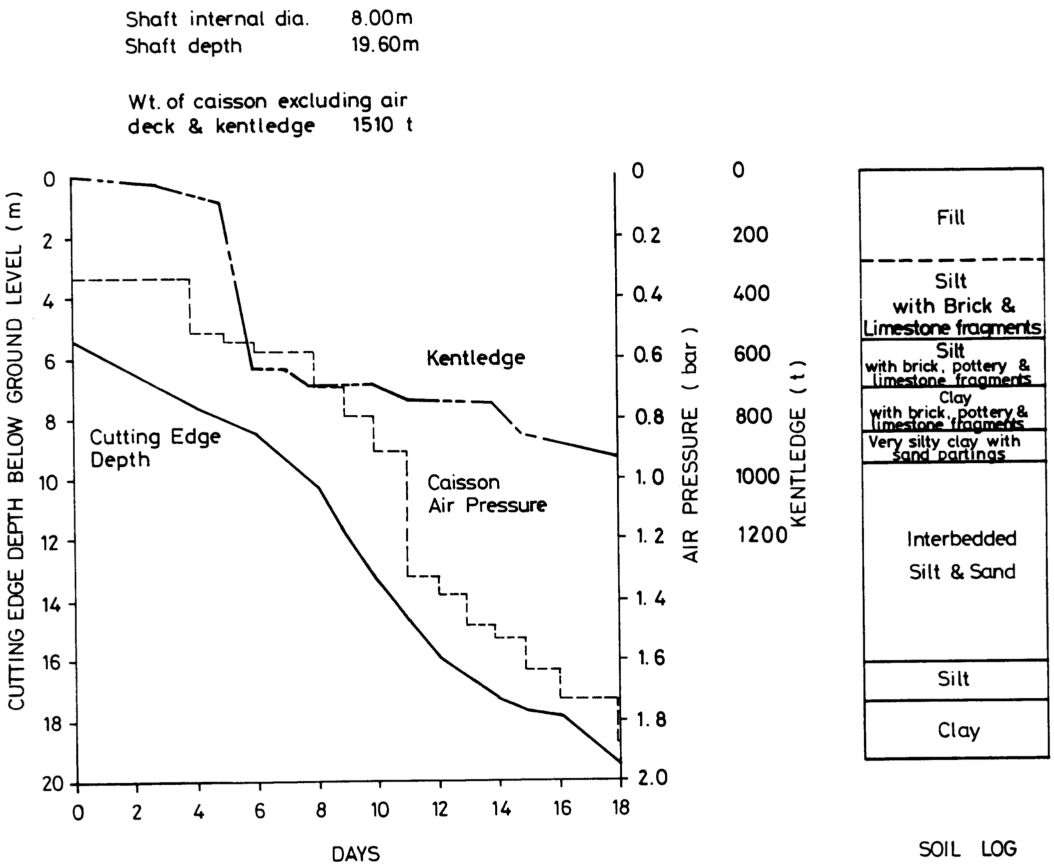
SHAFT 5 SINKING RECORD

Figure 11



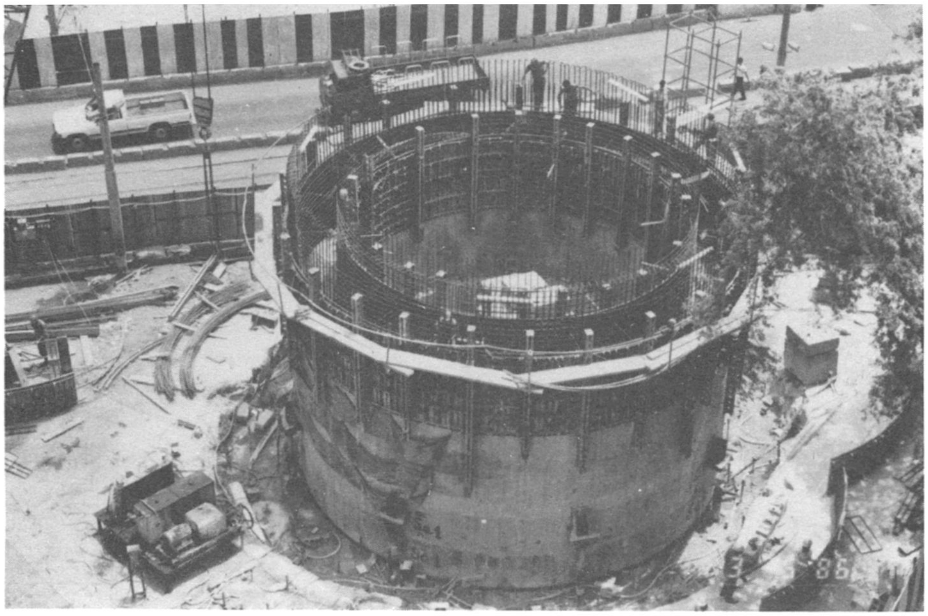
SHAFT 5A1 SINKING RECORD

Figure 12



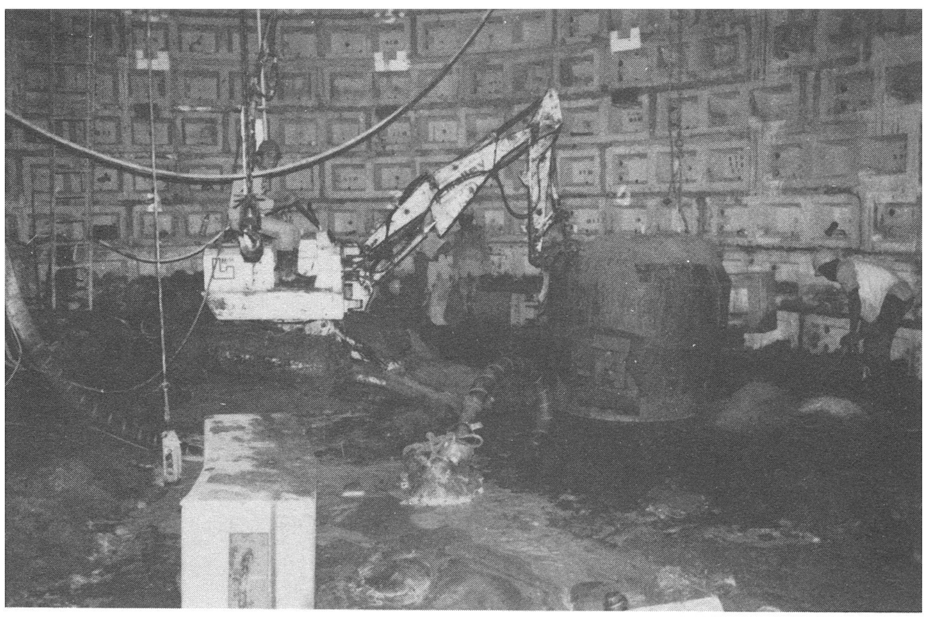
SHAFT 16 SINKING RECORD

Figure 13



CONTRACT 4

CAISSON CONSTRUCTION IN PROGRESS WITH EXTERIOR JACKING SYSTEM, SHUTTERS IN PLACE AND REINFORCEMENT BEING PLACED



CONTRACT 4

BOLTED SEGMENTAL LINING AND EXCAVATION INSIDE CAISSON USING EXCAVATOR AND LARGE SKIPS

Figures 14&15

Construction of the inline pump station, Milwaukee, Wisconsin, U.S.A.

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S.A. Healy Co., McCook, Illinois, U.S.A. (formerly Cementation Co. of America, Inc.)

SYNOPSIS

The Inline Pump Station contract was a $10M project that formed an integral part of the Milwaukee Waste Pollution Abatement Program.

The contract involved the sinking of three shafts through saturated overburden and dolomite limestone, and the construction of a large chamber and connecting tunnels.

This paper describes in detail the variety of underground construction techniques required, including shaft freezing, drill and blast excavation, grouting, shotcreting, concrete lining and large diameter pipe installation.

DESRIPTION OF CONTRACT

The Inline Pump Station Contract was bid in August, 1984 and won by the Cementation Company of America Inc., with a bid of $10,012,133. The Engineer's Estimate was 13% below the low bid. The time allowed for completion was two years. The works had been designed by Howard, Needles, Tammen & Bergendoff and contract administration was by CH2M Hill. The owner was the Milwaukee Metropolitan Sewerage District.

The contract was located adjacent to the existing sewage treatment plant on Jones Island, about one mile south of downtown Milwaukee on the shores of Lake Michigan. The worksite was in the midst of an area of intensive activity related to the Waste Pollution Abatement Program and was somewhat limited in size,

The various elements of the work were as follows:

1. Screening Shaft: 107m (350') deep with 6m(20') diameter in-situ concrete lining and internal divider wall; stub connection to 9m(30') diameter Crosstown Interceptor tunnel and connection to 3m(10') diameter Suction Header tunnel.
2. Ingress/Egress Shaft: 107 m(350') deep, 6 m(20') finished diameter with connection to Pump Chamber.
3. Equipment Shaft: 94 m(310') deep, 6 m(20') finished diameter terminating at roof of Pump Chamber.
4. Pump Chamber: 18 m(60') wide by 12 m(40') high by 34 m(110') long with reinforced concrete walls and shotcrete roof.
5. Suction Header: 82 m(270') long tunnel, incorporating 3 m(10') diameter prestressed concrete pipe liner feeding into three 14 m(45') long tunnels with 1.4 m (4.5') diameter stainless steel liners.

The Inline Pump Station is at the terminus of the Inline Storage System. This is a system of tunnels designed to provide emergency storage of combined sewage and storm water during periods of excessive rainfall. When construction is complete, material will flow into the Screening Shaft where it will pass through coarse screens into the Suction Header Tunnel. From here it will be fed to pumps located in the Pump Chamber, which will pump it to the surface treatment plant.

Notice to Proceed was issued on January 3rd, 1985 and mobilisation commenced immediately. The program envisaged commencing with the Screening Shaft, then the Equipment Shaft and finally the Ingress/Egress Shaft. The Suction Header and Pump Chamber were to be done once access was established

GEOLOGY

The stratigraphy at the Inline Pump Station site was as follows:

0–5 m (0–16')	Fill
5 m–8 m (16'–26')	Alluvial Deposits Silty to gravelly sands, medium dense to very dense.

8 m–25 m	Estuarine Deposits
(26'–8')	Layers of medium stiff to stiff silt interspersed with layers of medium stiff clay.
25 m–26 m	Alluvial Deposits
(82'–85')	Dense sandy gravel
26 m–39 m (85'–128')	Lacustrine Deposits
	Medium dense to very dense silty sands with stiff clay interbeddings,
39 m–44 m	Ice Margin Deposits
(128'–144')	Silty clays and sands
44 m–47 m	Lacustrine Deposits
(144'– 155')	Silty sands, medium dense to very dense with thin clay layers.
47 m–51 m	Lacustrine Deposits
(155'–168')	Medium dense to dense glacial till with small to medium boulders.
Below–51 m	Dolomite
(168')	Dense, close to medium bedded with zones of numerous very thin shaley partings, vuggy at depth. Unconfined compressive strenths 140 Mpa (20,000 psi) average, 300 Mpa (44,000 psi) maximum.

The water table fluctuated seasonally but was generally about 4 m(13') below surface. Permeabilities in the overburden were around 10–6 m/sec and ranged from 10–9 to 10–5 m/sec in the rock. Individual flows in the rock measured upto 13 l/sec (200 gpm) and were usually associated with the vertical or near-vertical joint planes.

GROUTING

Pre-grouting of the shafts and Pump Chamber from surface was specified and was carried out at the direction of the Engineer. Pre-grouting of the Suction Header from surface was not specified. Additional grouting was directed from underground during excavation.

Surface Pre-Grouting

Pre-grouting of the shafts, which were to be excavated to a minimum 6.6 m(22') diameter, was carried out through holes on a 9 m(30') diameter circle. All holes were drilled to the rock and then advanced in 6 m(20') stages to a final depth of 111 m (365').

Initially, three primary holes were drilled at 120 degrees, followed by three secondary holes between the primaries. In order to check the efficacy of the cover, a centre tertiary hole was then drilled. At the Ingress/ Egress and Equipment Shafts, no further holes were deemed necessary. At the Screening Shaft, two further holes were installed and grouted, both within the limits of the excavation.

Pre-grouting of the Pump Chamber was accomplished with a grid-like arrangement of holes along lines 3.4 m(11') apart with holes spaced 8.2 m(27') apart along the lines within the Chamber. Half this spacing was used for the lines along the north and south walls of the Chamber.

The procedure for each hole was as follows. An NQ size casing was drilled through the overburden and about 600mm (2') into the rock. The casing advancer was then withdrawn and a BQ size drill string entered into the casing. The hole was then advanced 6m(20') in the rock and the drill rods withdrawn.

A packer was then set at the top of the stage and a water test conducted. The Lugeon value was calculated and used to determine if grouting would be necessary. If grouting was required, injection would start with a 5:1 cement mix, followed by successive thickening, until practical refusal had been reached. When the grout had set, the packer was removed and the hole advanced another 6 m(20'). Ultimately the hole was abandoned by filling to the surface with grout and removing the casing.

Drilling of the holes was carried out with either a truck-mounted Longyear HC44 or a track-mounted Acker MPV. Injection was with a diesel-hydraulic mixing and pumping unit incorporating a Moyno 6P6 pump. In general one hole could be drilled and grouted in around 60 hours. During the surface grouting program some 5,000 bags of cement were used. No chemical grouting was done.

Underground Grouting

Virtually no advance grouting was done in the shafts during the excavation, despite grounwater inflows increasing with depth. Some spot grouting was carried out in arrears.

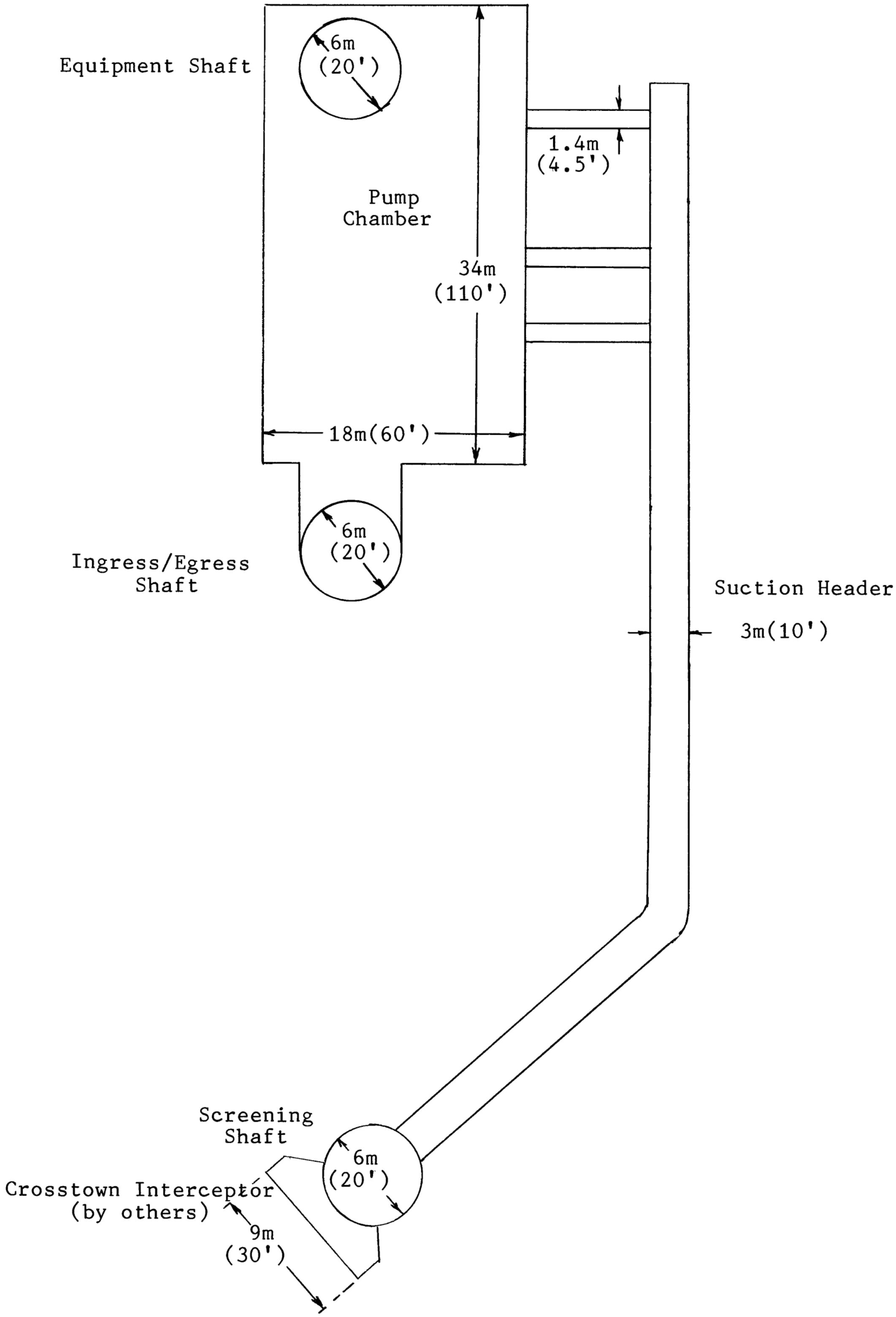

FIGURE I

LAYOUT OF INLINE PUMP STATION

The Suction Header tunnel was advance-grouted with a series of 15 m(50') long covers. These involved the drilling of four holes at the corners, which were then grouted. A centre test hole was then drilled and a further four holes drilled if required. The holes were grouted in one stage and all grouting was with cement.

A full grout curtain for the Pump Chamber roof was installed from a pilot drift. This was ordered due to the generally wet nature of the rock encountered in other parts of the excavation and the fact that the roof was to have a shotcrete lining. Fans of 9 m(30') long holes, 3 m(10') apart and with a 3 m(10') toe spacing were drilled. These holes were injected in one stage initially with cement. In an effort to dry up the roof as much as possible, chemical grouting was also carried out. Some 2,700 m (9,000') of drilling was required and about 1,000 bags of cement and 7,500 1(2,000 gals) of chemicals were consumed. Drilling was with track-mounted Gardener Denver PR125 pneumatic drills, consistently achieving 90 m(300') per shift. Injection was with Peroni piston pumps.

FREEZING

Because of the extent of saturated overburden and a prohibition on the use of dewatering, it was decided to freeze the shafts. Foraky (UK) were employed as freezing consultants.

Freeze holes were drilled around each shaft on a 10 m(34') diameter circle. There were 34 holes in all, of which half were drilled to depth 55 m(180') and half to depth 27 m(90'). The alternating of holes was to allow a rapid build-up of ice in the top part of the shaft, without a subsequent encroachment at depth. In addition, two observation holes were installed 1.2 m(4') and 2.4 m (8') outside the circle of freeze holes.

The holes were drilled by Test Drilling of St Louis using an Ingersoll Rand rig. Holes were drilled 180 mm(7") diameter with a tri-cone bit under mud containing 1.2 kg of bentonite per litre (10 lb/gal). A 6 m(20') conductor casing was used. Progress during the course of the work averaged around 6 m(20') per hour.

As each hole was completed, a 100 mm(4") diameter casing with a closed end was placed in the hole. When all the holes at the shaft had been drilled and cased, the deeper holes were surveyed using oil well techniques. This service was provided by both Eastman Whipstock and Sperry Sun. Surveying of the holes was required to establish if tolerances had been met. If two holes had diverged an unacceptable amount, the casing was removed from the intervening shallow hole and it was deepened to 55 m(180'). In rare cases, an additional 55 m(180') hole was drilled. The casings were also pressure tested for leaks.

Thereafter, a 50 mm(2") diameter polythene hose was placed inside the casing to act as a delivery tube with return up the annulus. The holes were connected up to 200 mm(8") diameter delivery and return headers and thence to the brine circulating unit. This had “warm” and “cold” compartments which actually differed in temperature by about 0.5C(1F). Three pumps each capable of 40 1/sec (600 gpm) provided circulation. All piping was insulated.

The system was charged with calcium chloride brine which was delivered to site as a 38% solution. This was diluted to give a solution with a specific gravity of 1.29. Cooling of the brine was achieved by passing it through two 64t(70T) refrigeration units, These comprised a heat exchanger using ammonia as the refrigerant, which was cooled by a 150kw (20hp) York compressor.

The two observation holes were equipped with thermocouples at vertical intervals of 6 m(20'), which were connected to a Honeywell recorder.

Once the system had been established, brine circulation began at a rate of 60 1/sec (1,000 gpm). Brine was quickly brought to below −30 C (−20 F). Closure of the ice wall was indicated by water rising in the central pressure relief hole. This took around 25 days. Development of the wall to design strength, as determined from observation hole temperatures, took a further 7 days. At this point excavation commenced.

Freezing continued during excavation and lining, with the flow of brine being regulated in an attempt to maintain the ice wall width without greatly increasing it.

Freezing of the Screening Shaft was discontinued in November, 1985. Readings taken at 55 m(180') depth in the observation holes, indicated that temperatures did not rise above freezing until March, 1986.

SHAFT CONSTRUCTION

Freeze Excavation

The basic method of excavation was with a 1.34 cu.m (1.75 cu.yd.) clamshell, suspended from a 45t(50T) crawler crane. Hand-held pneumatic breakers were used to trim the walls. As the ice encroached further into the excavation, a Kubota KH28 mini-backhoe fitted with a hydraulic breaker was introduced to break up the major amount of the frozen ground. Ice encroached 600 mm (2') down to around 40 m(130') depth, increasing to about 1 m(3') at 43 m(140') and freezing all the way acrooss the bottom of the shaft at 46 m(150'). At these depths drilling and blasting was commenced with the backhoe being used to load into 2.5 cu.m. (3.3 cu.yd.) muck buckets. Drilling of blast holes was done with Victor air augers.

Liner plate and ring beams were used to secure the top 6 m(20') of the shaft and thereafter mesh and vinyl were pinned to the wall to guard against falling material. Insulating blankets were hung for about 12 m(40') below the liner plate to counteract the thawing effect of the sun. The shafts were sunk to 55m(180') before being concrete lined.

Excavation progress per eight-hour shift was around 1.2 m(4') in unfrozen ground, 600–900 mm (2–3') in areas with less than 600 mm (2') of ice encroachment and 300– 600 mm (l–2') with the majority of the shaft floor frozen. A standard crew consisted of a foreman and four miners with a crane operator and toplander on surface.

Rock Excavation

Excavation was by drill and blast using hand-held sinkers. Where the shaft was wet, half-sump rounds were taken. If conditions allowed, a full-sump was blasted. In general, full rounds were 1.2 m(4') deep, requiring 110 holes for a full-sump. Unigel explosive was used for the bulk of the holes and Hercosplit for the trimmers. Nonel half-second delay detonators were used for initiation A powder ratio of around 1.6 kg/cu.m. (2.75 lb/cu.yd.) was achieved. Holes were drilled 40 mm(1−5/8") diameter with cruciform bits and hexagonal steels averaging around 90 m(300') of drilling per unit.

Mucking was generally with the Kubota, although an Eimco 630 was also used. The Kubota could load a 2.5 cu.m. (3.3 cu.yd.) bucket in four minutes compared to seven minutes for the Eimco 630.

Support was by the use of 1.5 m(5') long split-set rock bolts arranged on a 1.2 m (4') pattern. Mesh was added by the use of 450 mm(18") split-set inserts where required.

Crew make-up was the same as for frozen excavation with three eight-hour shifts being worked each day. A typical cycle for a 1.2 m (4') advance was as follows:

Drilling	7.0 hrs
Loading	2.1 hrs
Smoke	0.5 hrs
Mucking	8.2 hrs
Support	2.4 hrs
Services	1.1 hrs
Delays	2.7 hrs
Total	24.0 hrs

Concrete Lining

Lining of the shafts was generally done in two lifts. Initially the shaft was lined from 55 m(180') depth to surface prior to turning off the freeze. Upon reaching final depth, the remainder of the shaft was lined.

Lining was carried out with a 6 m(20') high steel form, working from a three-deck stage. The form was hung from the stage with four chainfalls. The stage was suspended from two 40 kw(50hp) New Era winches, each with a 23,000 kg (50,000 lb) line pull. Concrete was lowered into the shaft in a 1.2 cu.m. (1.5 cu.yd.) bucket and placed in the form through elephant trunking.

Design thickness for the concrete was 450 mm(18") in frozen ground and 300 mm (12") in rock. The shaft was excavated to 7.2 m (23.5') in frozen ground and 7m (23') in rock. In each case, the average thickness of concrete was 600 mm (24"). Around 80 cu.m. (100 cu.yd.) of 28 Mpa (4,000psi) concrete and 640kg (1,400 lb) of rebar were needed for each pour. Joints were built with 150 mm (6") deep waterstop.

Using a standard mining crew working three shifts, it was possible to achieve a pour per day. Additional time was needed to set up the operation and further installation of panning in wet areas. Patching and contact grouting was carried out as a separate exercise towards the end of the contract.

Screening Shaft Internal Divider Wall

The divider wall was required to allow screening for the material entering from the 9 m(30') diameter tunnel, to provide guideways for a clean-out clam to be lowered from surface and to provide a housing for a surface overflow pipe. As such, it was very intricate.

It was formed by using a 4.6 m(15') high steel form built by Economy Forms, using a combination of standard and special panels. The form had to be broken down into four sections for each move. The work platforms were integral with the form.

The wall included 12 steel guides, a 900 mm (36") diameter steel pipe and very dense rebar, inhibiting the ability to vibrate internally. Superplasticised concrete was therefore used to ensure a satisfactory finish. The design quantity of concrete for each pour was 28 cu.m. (36 cu.yd.) with actual usage indicating a 7% wastage factor.

The standard crew was used with the addition of two carpenters. After a very long learning period, production reached one pour every 40 hrs.

SUCTION HEADER CONSTRUCTION

The Suction Header was designed as a length of 3 m(10') diameter conduit, feeding into three 1.4 m (4.5') diameter conduits. The 3 m(10') section was to be lined with pre-stressed concrete pipe and the smaller sections with stainless steel pipe.

Suction Header Tunnel Excavation 3m (10')

As the concrete pipe was 3.6 m(11'8") external diameter, it was decided to mine the tunnel as a 4 m(13') modified horseshoe. Drilling was with a Joy, crawler-mounted two-boom pneumatic jumbo. Sixty holes, 3 m(10') long, were drilled per round and these were loaded with a combination of Unigel and Hercosplit and initiated with Nonel detonators. Explosives consumption was 3.9 kg/cu.m. (6.8 lb/cu.yd.) and the average advance was 2.7 m (9'). Drilling was with 45 mm (1–3/4") diameter button bits averaging 90 m(300') per unit and 25 mm(1") round steels averaging 170 m (550') per unit.

Mucking was with a 1.2 cu.m. (1.5 cu.yd.) International crawler loader dumping directly into muck buckets at the Screening Shaft. Support was provided by split-set roof bolts as required.

The excavation was continually interrupted by having to stop to grout and by the presence of large flows of water. Towards the end of the drive, water became less of a factor and a typical cycle for a 2.7 m (9') advance was as follows:

Drilling	7.9 hrs
Loading	1.9 hrs
Smoke	0.9 hrs
Mucking	6.4 hrs
Support	0.2 hrs
Services	1.2 hrs
Delays	3.3 hrs
Total	21.8 hrs

Pipe Installation 3m(10')

The concrete pipes were supplied by Lock-Joint Co of New Jersey. They were 3 m(10') long and weighed over 18t(20T). They incorporated a double gasket, bell and spigot joint.

A purpose-built, hydraulically-operated pipe carrier was obtained for the transportation and placing of the pipes in the tunnel. This consisted of a long needle beam with wheels and outriggers at either end and a set of bull wheels at the front. As the carrier ran on track, a sub-invert was poured, incorporating 100 mm(4") angle-iron as track sections. These were set to line and grade so that the pipe could be laid directly on them.

The first pipe to go into the tunnel had an integral bulkhead which prohibited the use of the carrier. For this pipe, a steel cradle was fabricated which ran on Hillman rollers. The pipe was lowered down the shaft with a 145t(160T) American crawler crane and pushed into the tunnel with a Caterpillar 910 loader. Once in position, jacks were used to lower the pipe and cradle onto the rail. The cradle was left in place and the rollers withdrawn.

The procedure for the remaining pipes was to lower them to the bottom of the shaft and thread the carrier through. The pipe was then pushed to within 1.5 m (5') of the previous pipe and the front bull wheels lowered into the previous pipe. The front carrier wheels were raised and the pipe pushed to the point where the joint was about to close. The front and rear outriggers were then lowered, the front ones inside the previous pipe and the rear ones onto the tracks. The pipe was then lowered onto the track and pushed home with jacks.

This operation originally took a number of days due to the the gaskets on the pipe being oversize, making joint closure difficult. Once this had been corrected it was possible to install a pipe in eight hours.

Backfilling of the pipes was done in 25 m(80') lengths. Slick lines were placed to the exterior of the pipes and the end pipe bulkheaded. A 100 mm(4") diameter delivery line was installed from surface down the shaft and into the tunnel. A specially designed 14 Mpa (2,000psi) backfill concrete incorporating a high percentage of fly-ash, was used. This was discharged from ready-mix trucks on surface into the delivery line and flowed under its own head through the delivery and slick lines and around the pipe. No problems were experienced with blockages.

Tunnel Excavation and Pipe Installation 1.4 m(4.5')

These tunnels were driven as 2.4 m(8') square headings from the Pump Chamber using jacklegs. Mucking was with an Eimco 630 dumping directly onto the Chamber floor. These three headings were a total of 40 m(135') in length and were excavated in 17 shifts.

The 1.4 m(4.5') pipe liners were supplied by Progressive Fabricators of St Louis as a straight and elbow section, which bolted together and onto the concrete pipe. Installation required no special equipment. Timbers were laid on the floor of the heading and the elbows were pulled in with a chainpull. The straight sections were pushed in with a loader. Each heading required 7 shifts for pipe installation. Backfilling of the pipes was done with a Conspray concrete pump, using exterior slick lines.

PUMP CHAMBER CONSTRUCTION

The Pump Chamber was designed with an elliptical roof, curved sidewalls and flat end walls. Resin dowels, 5 m(16') long and 25 mm(1") in diameter, were required at 2.4 m (8') centres in the roof and walls.

Excavation

The Chamber was opened up with a 5.5m (18') wide by 3.7 m(12') high pilot heading driven along the roof from the Equipment Shaft to the Ingress/Egress Shaft. This was excavated with the Joy jumbo and the Caterpillar 910 loader. The pilot heading was used for the grout cover and the installation of extensometers and dowels. Thereafter, the roof of the Chamber was opened out to full width and the remaining dowels and the shotcrete lining installed.

The bulk of the Chamber was excavated in three lifts from a sinking operation at either end. Blast holes were generally drilled horizontally with the Joy jumbo and loaded with Unigel. Mucking was with the Caterpillar 910 into a 7.3 cu.m. (9.3 cu.yd.) muck bucket.

The excavated volume of the Chamber was 6,100 cu.m. (7,800 cu.yd.). Drilling averaged 2.4 m per cu.m. (6'/cu.yd.) and explosives consumption was 1.6kg/cu.m. (2.8 lb/cu.yd.).

Because of the necessity to carry out lining and support work during the excavation phase, there were frequent interruptions. However, over one three-week period that was relatively free from interruptions, 1,600 cu.m (2,000 cu. yd.) of rock was excavated at an average of 35 cu.m. (45 cu.yd.) per eight-hour shift. During the same period, 65 resin dowels were installed at 0.6 hrs each.

The crew times in hours per cubic metre (cubic yard) were typically as follows:

Drilling	0.051	(0.04)
Loading	0.025	(0.02)
Smoke	0.013	(0.01)
Mucking	0.076	(0.06)
Support	0.013	(0.01)
Services	0.013	(0.01)
Delays	0.038	(0.03)
Total	0.229	(0.18)

Shotcrete

Despite the extensive grout cover carried out from the pilot heading, the roof of the Pump Chamber was still very wet. In order to improve adhesion of the shotcrete, it was decided to use silica fume as an additive. Shotcrete was delivered to site in ready-mix trucks and the silica fume added as a liquid. The mix was then lowered to the Chamber in buckets and discharged into a Conspray pump.

Application was at the rate of 15 sq.m. (18 sq.yd.) per hour for a nominal 50mm (2") thick layer. Due largely to the wet conditions, rebound was excessive, with the volume of material delivered to site being twice that measured in place. Actual throughput was around 1.6 cu.m. (2 cu.yd.) per hour.

Concrete Lining

The Chamber end walls were formed with timber panels and a proprietary waling system supplied by Patent Scaffolding. The curved side walls were formed in four main pours using 15m(50') long timber forms manufactured by Custom Form. Concrete was delivered into the form with the Conspray pump.

A sidewall pour required about 80 cu.m. (100 cu.yd.) of concrete and 450kg (1,000 lb) of rebar and took around eight shifts to complete.

CONCLUSIONS

The Inline Pump Station contract involved a wide variety of construction techniques, many of them new to the Milwaukee labor force. This meant a longer learning curve in some instances before an acceptable level of production was reached. No time was lost due to disputes.

What problems there had been were mostly associated with water. In the Screening Shaft, an inrush of water through the pressure relief hole during excavation in the freeze, necessitated the taking of remedial measures with a three month delay. Excavation in the rock was constantly hampered by high water flows, with upto 13 l/sec (200 gpm) being pumped from the shaft on a regular basis.

Freezing of the Ingress/Egress Shaft was delayed due to the pumping activities of an adjacent contractor causing flowing groundwater conditions which increased the freeze time by 16 days.

Despite these difficulties, the contract had its successes, particularly in the area of shaft sinking. The mini-backhoe proved to be a versatile and productive alternative to conventional shaft muckers.

The project was completed in March, 1987 26 months after commencement. With extensions of time, this was well inside the contractual completion date.

Overview of current South African vertical circular shaft construction practice

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SYNOPSIS

The introduction deals with trends in the South African underground mining industry and its demand for shafts. The development of the modern subsidiary sinking industry and the depths.and dimensions of shafts which are being predominately constructed.

A description of all the phases and activities involved in the construction of a typical shaft follows. Commencing with the construction of the collar and pre-sinking, the establishment of plant, equipment and gearing up of a project to the commencement of fast sinking operations.

The paper then deals with sinking routines, i.e. lashing, blow over, drilling and blasting are detailed and comment is made on modern developments in mechanized drilling, watergel based explosives and electro magnetically initiated detonators.

Activities associated with sinking are then described, i.e. cover drilling, curb-ring and lining placement, station construction, initial development and loading pocket construction.

The excavation and lining phases completed, the paper deals with shaft steelwork preparation, checking, set jigging and referencing and guide drilling.

Text on the equipping phases follows. The up run in which the shaft is stripped of sinking services and brattice walls installed. Headgear changeover to equipping condition, equipping of the shaft with furnishings and the final changeover to permanent conditions.

It is concluded with a glossary of local shaft sinking terms.

INTRODUCTION

The South African underground mining industry

This is a very large industry which produces ores of virtually all the precious and base metals, coal and other minerals such as diamonds and asbestos. In recent years, depressed base metal, industrial mineral and coal markets have resulted in very little investment in new production capacity in these areas.

This has not been the case in the gold and platinum group metal sectors of the industry which have enjoyed better market conditions, thus stimulating exploration and investment in new mines and in replacement shafts on existing properties. These mines generally exploit fairly narrow reefs. The extensions of existing mines and the new areas being developed are tending to reduce in grade and increase in depth. This means that more shaft is required per kilogram of metal produced and per square metre stoped. As long as this trend continues shaft sinking will remain a routine, commonplace operation in this country.

Shaft sinking

This activity has progressed over the last 30 years, from an arcane art which was dominated by a handful of powerful personalities who were employed by and sank for the various mining houses, into a mature service industry.

During the past 20 years there have usually between 30 and 40 shafts in the course of construction at any one time. Of these about 75% have been constructed for gold and the balance for platinum, coal and base metals, minerals and diamonds. The gold sector has retained its dominance and platinum now enjoys the lion’s share of the balance.

The volume and stability of demand has encouraged the growth of a substantial specialist contracting industry. These contracting companies, some of which are controlled by the mining houses account for about 70% of the sinking currently being carried out in the country, the balance being conducted by the mines and mining houses themselves. One mining house in particular does virtually all its own sinking and fields several crews, which it has scheduled to keep occupied for the medium term future.

The contractors have a symbiotic relationship with the mining industry. The establishment, and growth of these companies over the last 30 years into free standing enterprises has been to the benefit of the industry as a whole. They have become storehouses of experience, expertise, personnel, plant and equipment which can be mobilized at short notice to the convenience of the industry. The contractors compete against each other for work and the industry can call for firmly costed tenders for complex projects virtually as required. In the market in which they operate the client is normally capable of carrying out his own sinking. This, together with the pride they have in their individual products and reputations, is conducive to a healthy, competitive business environment. This in turn leads to the effective control of costs and has enabled the containment of cost increases in shaft sinking to levels below the working cost rises experienced by the industry.

Safety in sinking shafts

The harsh environment, the overwhelming noise and the controlled violence of many of the operations combine with the force of gravity to create a climate in sinking shafts that had traditionally been regarded as dangerous.

This is no longer the case. The size and relative stability of the sinking sector has generated a core of experienced men who build careers and spend their whole working lives in shaft construction.

The achievements of these professionals in the field of safety are more impressive than the products which they create and in which they take such pride. There are several instances now on record where a shaft has been completed without a fatality. Fatality free runs in excess of 200 000 shifts are becoming commonplace (a conventionally sunk typical shaft, complemented for a 7 day working week, clocks in around 10 000 man shifts a month.) Indeed accident rates in shafts are now comparable with the underground mining industry in general. This is the result of vigilance, imagination and effort exercised by every person involved in the great team effort that all shaft sinking requires. The efforts of these individuals are co-ordinated into programmes which command the commitment of both management and crews.

Elements common to all programmes are; unyielding discipline, co-operation and mutual protection between individuals, use of standard procedures, record keeping, competition between shifts and sites. Co-operation between the sinking organizations, vigilance and reaction to observed hazards, incentives, meticulous maintenance and control of equipment and immediate investigation of accidents all lead to the implementation of preventive action and a willingness to accept any change in method, design or protective gear that will contribute to improved safety.

These attitudes and efforts have been rewarded. A youth can enter the industry and anticipate with confidence that he will be able to retire with his health unimpaired, after a fulfilling, exciting and rewarding career in what has become prestigious profession.

The shafts

The circular concrete lined vertical shaft has ousted other cross-sections. Rectangular and elliptical shafts sometimes feature as the subject of debate but are only seriously considered if an existing shaft of such a cross-section is to be deepened. One other shape is occasionally used, this is the stretched circular, where a short flat, usually less than 10% of the shaft diameter, is introduced between 2 semi circles. The introduction of the flat offers some advantages in geometry and set design, this must be weighed against the impeachment of the integrity of the circle and the weakness so introduced into the shaft shutter.

Finished diameters commonly vary between 6 & 9 metres the most popular being around 8 which is a convenient compromise between construction and permanent configuration constraints. Shafts outside these dimensions would be considered very large or small and may require special construction techniques to be introduced. Depths of between 1500 and 2000 m are considered normal and some ultra deep single lift shafts of depths approaching 2500 m have been sunk and are in service. The ever increasing depth of exploitable reefs has resulted in shafts of some 3000 m being included in feasibility studies for future mines. This has initiated concerned debate amongst stakeholders in the shaft sinking community, who are considering ways and means of extending and adapting their technology to accommodate for these depths.

Depths of this order are straining the limits of rope technology. Ultra high tensile steels are required for both stage and kibble ropes. Ropes constructed of these materials are difficult to handle and it is very much a case of learn while you earn. Some disappointing results have been experienced with these ropes, but manufacturers are claiming to have overcome most problems and maintain they now have a reliable and predictable product. Rope weight to final load/pay load ratios are large, indeed most of the power used in these winding systems is dissipated in overcoming their own inertia. It appears therefore that a breakthrough in rope technology is a pre-requisite before any quantum increase in single lift shaft depths can take place.

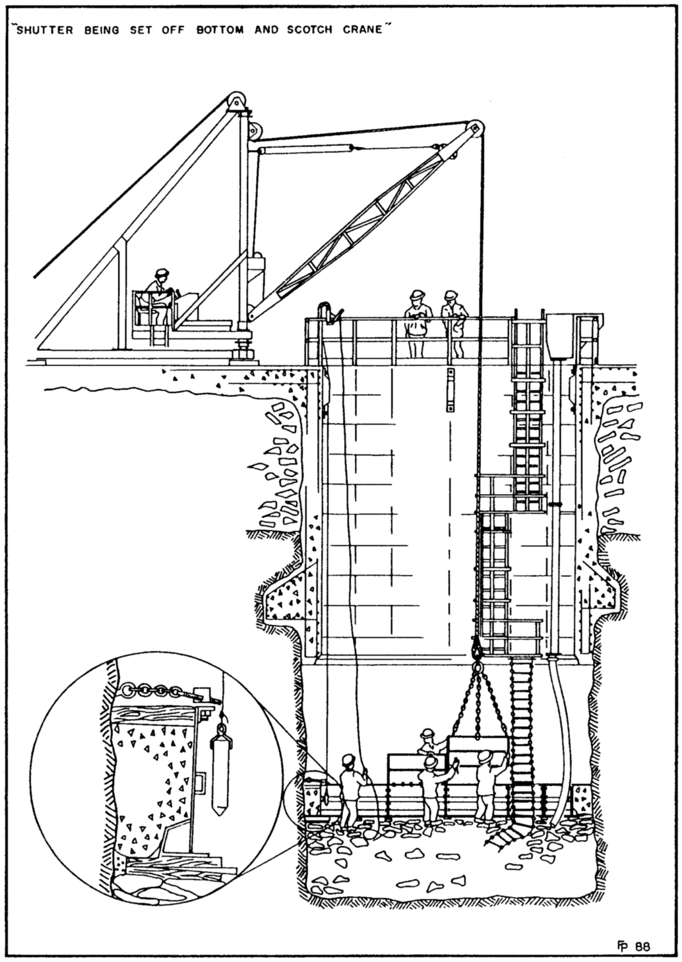

FIG 1 COLLAR CONSTRUCTION

COLLAR CONSTRUCTION AND PRE-SINKING

In the case of a steel permanent headgear having been selected both collar construction and pre-sinking can be effected prior to headgear erection and they can take place while the headgear is being manufactured.

Where a temporary sinking headgear is to be used, it can be erected after collar completion and employed for the pre-sink and the use of cranes or derricks for hoisting dispensed with.

When a concrete headgear is chosen, the collar is constructed through the headgear foundation raft and the pre-sink is usually carried out once the headgear slide has been completed.

Collar construction

The upper portion of the shaft collar is usually constructed in open cut by civil engineering techniques and backfilled. The shaft shutter is the first item of equipment ordered and is used in this construction. Where self supporting soils are to be traversed a back hoe may be employed to excavate to the limit of its reach, about 6m below surface. The bottom can then be levelled off and the shutter built from this floor. Reinforcing is placed as required and the barrel poured and extended hand over hand back to surface or to the civil section of the collar. Excavation can then proceed in pickable materials with the aid of pneumatic moils, hand tools and the probable assistance of a pneumatically powered crawler mounted rocker shovel. What is considered to be a safe sidewall height may be so exposed (ideally this height will coincide with the designed shaft lining lift length and set interval.) The shaft shutter can then be broken out lowered, assembled, reinforcing extended and the lining poured. Sinking is interrupted to permit these procedures. This process is then repeated until fresh rock is encountered and blasting commences. At this stage the collar is usually toed into a hitch cut in sound rock. Reinforcing is discontinued at this point. The collar section is then complete and pre-sinking operations may commence.

In rare cases when the ground in which the collar is sited was not self supporting, special techniques have been used. The unstable section has been traversed using caisson shielded sinking methods or the area has been pre-supported with closely

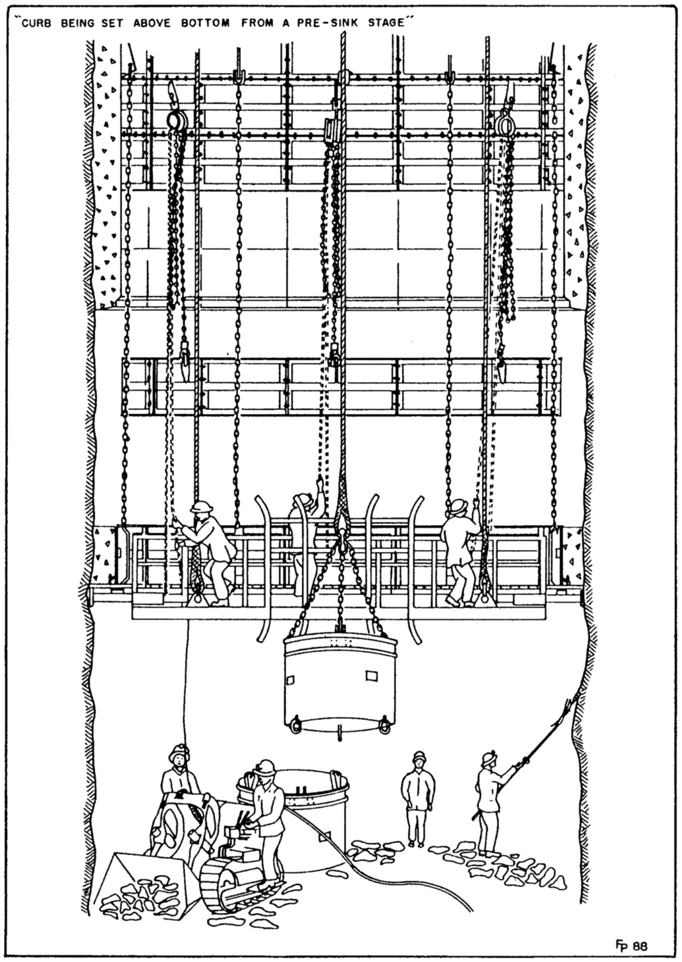

FIG 2 PRE-SINK

spaced auger drilled concrete piles which were founded at auger penetration refusal depth. The pile caps were connected into a reinforced concrete ring beam so that excavation could proceed within a structurally sound pre-formed cage.

Ground water inflows can be problematic in unstable measures. These are usually handled with pumps or prevented from entering the excavation by diversion to well points sunk around the shaft site from which the area can be dewatered.

Hoisting arrangements for this phase of operations can be of various types. Popular methods are:- mobile cranes, stationary scotch cranes with a swivelling derrick and slewing jib or arrangements of fixed or slewing gantries with load carrying traversing crawls. They should be capable of handling the afore mentioned rocker shovel. If their use is to be extended for the presink, arrangements for guided travel must be incorporated into their design when this becomes mandatory i.e. once the shaft has advanced more than 6 times the shaft diameter.

During collar construction, sinking and lining operations are usually sequential and the shaft shutter is built off the bottom, with access to the shutter usually provided by scaffolding. The shutter is lowered with crab winches sited around the shaft and mounted on the bank.

Pre-sink

The object of presinking is to construct sufficient depth of shaft, to permit the assembly of the sinking stage and lashing unit in the shaft bottom. Another requirement is to open up adequate clearance between the shaft bottom and the stage parking position to allow blasting without damaging the stage or the more vulnerable lashing unit. A bottom to bank interval of say 90 m would be ideal. This would allow about 70 m between the shaft bottom and the underside of the grab driver's cab and about 20 m for the stage. Utopian conditions however rarely pertain and the pre-sink is usually not so deep. This situation then demands that shorter lightly charged rounds are pulled until a safe stage withdrawal height is obtained.

The main difference between collar construction and pre-sinking is that in pre-sinking the curb ring is suspended and lining can be placed some distance above the bottom, so that sinking and lining can be carried out concurrently.

A stage is required for shaft bottom protection and for access to the shutter. A specifically built presink stage is normally introduced to fulfill this need or in some cases the top 2 decks of the main sinking stage may be employed for the purpose.

Hand held drill and blast techniques are used for rock breaking and this section of shaft serves as a useful training period for the shaft crews. Lashing is generally effected with crawler mounted rocker shovels. When the pre-sink depth is attained sinking is interrupted and the main sink stage assembled either on the bottom, or pre-erected on surface alongside the shaft and lowered complete onto the bottom, with a large crane. The stage is roped up and raised, the lashing gear installed, commissioned, and the shaft stripped of pre-sink services and equipped with pipes etc. ready for the main sink to commence.

The lead time involved between the start of on site work on the project and this stage of maturity is at least 6 months, during which time the following would have been achieved:

1. Crew accommodation established
2. Power, water and compressed air supplies secured and the site reticulated with these services.
3. Pre-sink completed and this plant and equipment cleared from site.
4. The main sink kibble and platform hoists erected and commissioned.
5. Shaft concrete batch plant erected and commissioned.
6. Headgear, tipping and muck disposal arrangements, bank doors and banksman's control cabin erected and in working order.
7. Offices, workshops, changehouses, garages, stores and all other site buildings erected and occupied.
8. Stage, lashing unit and all in-shaft services ready to go.
9. Supplies of permanent and consumable materials secured and deliveries scheduled.
10. The site adequately staffed and equipped for sinking to proceed.

SINKING—ASSOCIATED ACTIVITIES AND LINING

Sinking

The effectiveness of this activity governs the productivity of the entire operation, for this reason the shaft bottom crews are paid the highest bonuses. Ideally the lash, drill, charge up and blast cycle should be completed within eight hours in conventionally sunk shafts. This is the over-riding consideration when selecting and deciding:

Kibble winder capability
Kibble sizes
Crew strengths
Round lengths
Muck disposal arrangements
Lashing unit and grab sizes.

Typical effective round lengths range from 1,5 to 2 m with corresponding uninterrupted daily calls of 4,5 to 6 m.

Most sinking in this country is carried out on three shifts and what is termed "Call Out." In this system the following shift is called when the explosives are sent down on the current shift. The complete blast to blast cycle is then placed in the continuous control of a single crew. This provides the strongest possible incentive to complete the cycle in the shortest achievable time. It introduces the principle of accountability because an identifiable entity is responsible for all aspects of the round and other work executed on that particular shift. The system likewise fosters a high degree of competition between shifts for the shortest cycle times, another excellent motivator.

Lashing

The use of turret type lashing units and cactus grabs is almost universal. The units are matched with 0,56 cu.m grabs in shafts of between 6 and 8 m diameter; larger units and 0,85 cu.m grabs are used in the larger shafts. Crawler mounted rocker shovels have been used in very large diameter shallow shafts and in small shafts down to 5,5 m excavated diameter. Small, 0,28 cu.m remote mounted grabs have likewise been employed. In very small shafts hand lashing is sometimes considered, this is costly, rare and very unpopular nowadays and it is more usual to set the minimum shaft size at that which will accommodate a mechanized cleaning system for reasons of efficiency and economy.

Normal practice is to re-enter the shaft shortly after the shots have detonated and travel through the "plug" of smoke. Most shafts are force ventilated by surface mounted fans delivering air to the stage and below through one or two wall-mounted shaft ventilation columns. The stage and lashing crew ride the stage into the bottom, watering and barring down the side wall, and

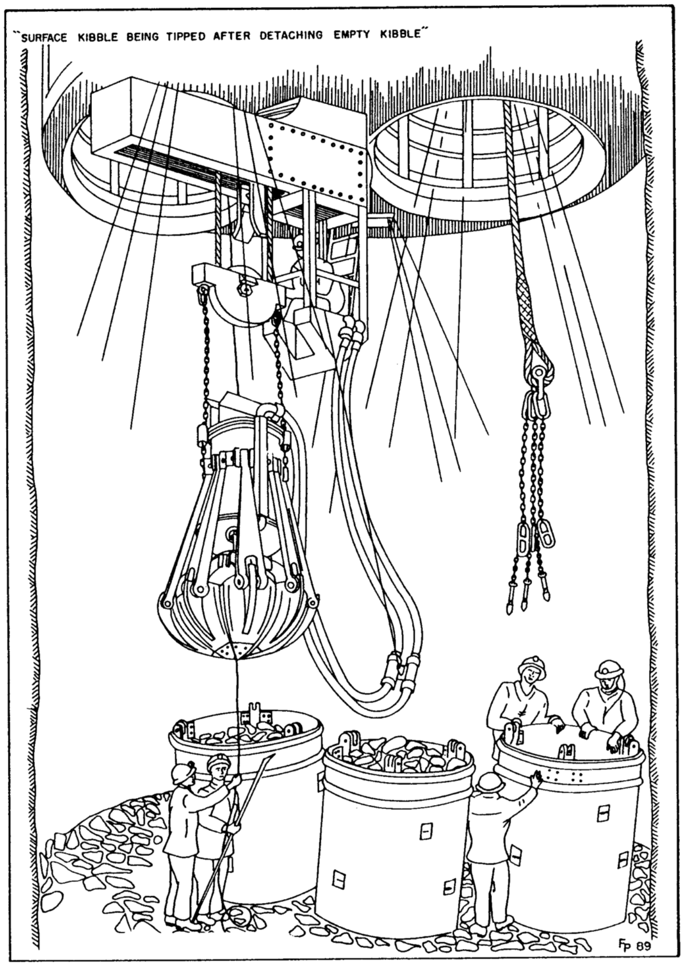

FIG 3 GRAB LASHIN

then bring it up to lashing elevation some 13,5 m above the muck pile and steady it, with the stage jacks against the shaft wall.

The grab driver levels the shot rock and the kibble winder is clutched between the muck pile and the tip. The first kibble is parked on the pile and the chains rung clear. The next kibbles bring down more crew and lashing begins with two kibbles in the bottom and two on the ropes. Progress is considered satisfactory if the first kibble of rock is out and tipped within the first hour. Cleaning settles into a rhythm with productivity peaking in the second or third hour. The empty kibble comes down, is sited and unhooked, the loose chains are rung clear and the opposite side kibble is tipped. When the chains come down the full kibble is hooked, raised, steadied and rung away.

Strict discipline must be maintained during this portion of the cycle and personnel travel restricted to down going trips which are less disruptive to the operation. During the third and fourth hours water may appear in the muck pile and could require simultaneous cleaning, pumping and baling. Hand lashing labour will be required about three kibbles before the end of the cleaning cycle and the "blow over" will commence. At this time solid rock will have been exposed and two×50 mm air hoses are lowered into the bottom, manned and muck blown into piles for the grab to remove. The sinker conducts his examination during this period and locates sockets and misfires which are piped out and examined. The bottom is dressed and barred to solid, the sinker gauges the amount of residual dirt in the bottom and signals for the detachment and parking of kibbles on the bank. As the bottom becomes cleaner the effectiveness of the grab reduces and final cleaning is done by hand with shovels, rock is lashed into the grab through the gaps between the top of the tines. By now the entire drilling crew should be in the bottom, ready to receive the machines as soon as the machine kibble arrives, by which time the bottom should be solid and table top clean. The lashing unit boom is slewed to parking position between the stage kibble holes, the grab raised and tied to the boom with a safety sling. The unit is then serviced and readied for duty on the next shift.

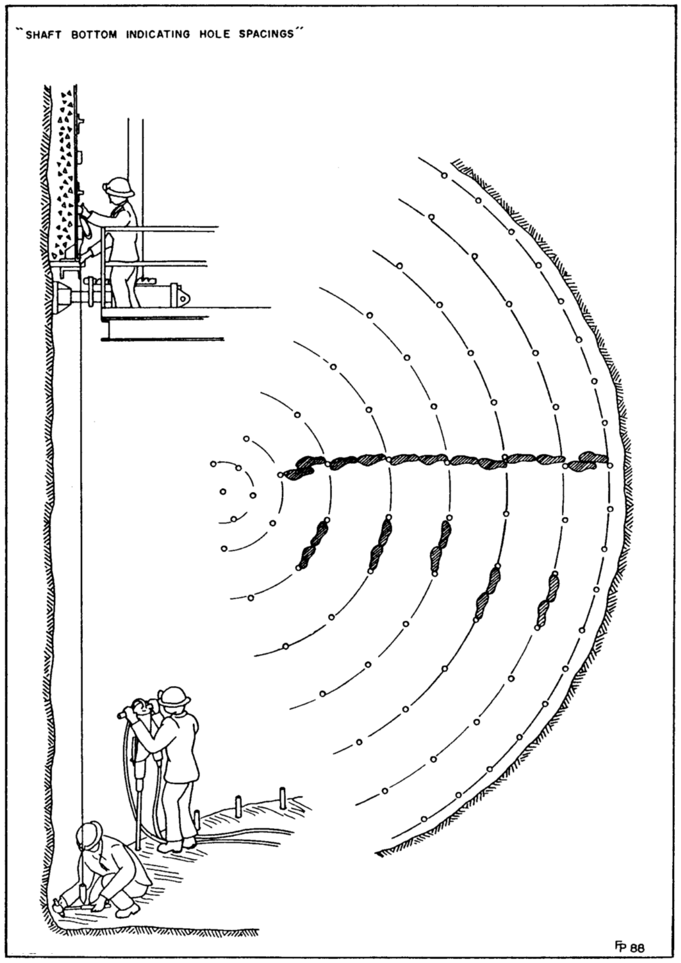

FIG 4 DRILLING

Drill, charge up and blast

Conventional hand held drills

The foreman and an assistant together with two machine operators and their light rockdrills go down the shaft with the first kibble of lashers. The assistant alights at the bottom deck of the stage and the rest on the shaft bottom. The assistant lowers a small plumb bob on a length of twine to the bottom, the upper end of the plumb line is held under the curb ring on the finished concrete line, thus transfering this position to the bottom. The foreman uses this as a reference and marks off a cropper and the downward vertical with paint or crayon making due allowance for the required concrete thickness. He then repeats the process at 0.5 m intervals around the circumference of the shaft. He is followed by the two machine operators who drill stub starter holes of about 100mm in depth. This activity is called scribing. The foreman will simultaneously check the shaft walls for clearance and arrange to have tight spots knocked off or popped off. This is especially important before the curb is lowered when shutter/sidewall interference will cause lengthy delays.

During this portion of the cycle the machine kibble with its underslung manifolds and hoses arrives in the bottom. The manifolds are pulled over to their park positions and the machine kibble's cargo of heavy sinker rock drills, drill steel and probably rock anchors is unpacked. The machines are connected up to the compressed air and water supply hoses and drilling commences. For a 2 metre round, a 2.5 metre long steel drilling a 2,3 metre deep hole is required. A drill steel of this length capped by a heavy rockdrill is about the maximum that can be handled safely by a fit, strong, well trained machine crew.

The machine and crew complement will be set (24 to 30 machines for a 180 to 240 hole round is normal) so that the bottom can be drilled out within about an hour. Each machine will be required to drill between 7 & 8 holes. About 4 machines will begin drilling on the cut and cut easers. This is usually under the supervision of the sinker and burn or cone cut configurations are popular. The rest of the machines start drilling on the croppers and work their way inwards while the cut machines work outwards. Ring burdens and hole spacings are determined by stepping off boot lengths. The machine operators concentrate on drilling while their assistants recover steel from the completed holes which are then piped out and stopped with socket plugs to keep them clean. Water is pumped into the kibbles which are now clutched between just below the bottom deck of the stage and the tip. Surface disposal is usually by siphoning.

As drilling nears completion the machine kibble again arrives in the bottom loaded with explosives and charging sticks. These are removed and the kibble repacked with drill steel and machines, the manifolds and hoses slung under the conveyance and sent out. The blasting harness is broken out and bus wires and detonators run around the shaft bottom adjacent to the matching rings of holes. Charging up likewise commences from the croppers and works inwards. The inert wax primer containing the inverted long period delay detonator is always placed at the bottom of the hole. 32 mm dia.×200 mm×60% ammon gelignite is almost universally used in hand drilled shafts. In the croppers two sticks are placed directly on the primer to clear the toe and initiate the cushion blast charge of smoothex loaded in omega clips or small diameter decoupled 100 mm long gelignite sticks placed 100 mm apart in omega clips. The balance of the round is charged with the 32 mm dia×200 mm gelignite sticks at the rate of 2 :3 charge length:hole length. The red bus wires are then all twisted together as are the black so that all the detonators are in parallel circuit and these are then connected to the hanging blasting cable. The bottom is cleared of personnel and the sinker travels with the last kibble to the top deck of the stage. The stage is then raised to blasting position, the blasting cable connected to the blasting terminals and the stage then cleared of personnel. The round is set off from surface by the sinker once the shaft is clear.

Current technological innovations

Drill Rigs

The development of the shaft drill rig is one of the most significant advances in shaft sinking technology in this decade. Rigs have been employed in about a dozen South African shafts. This technology can be regarded as proven and current models are well designed, workmanlike tools. The application of these drilling systems will probably become more commonplace. They will be of greatest benefit in the large deep shafts which are currently being contemplated.

The potential benefits of the rigs are in many instances in the process of being realized and are:

1. Enhanced productivity both in terms of units produced per man shift and improved absolute sinking rates.
2. Reduced costs per metre sunk.
3. Improved environmental and safety conditions due to reduced human and machine populations on the bottom resulting in a quieter, easier to manage work place.
4. Smaller crews.
5. Easier administration and control of crews on 7 day work weeks. This is because of the ease of introduction of 4 shift operations. A lashing shift and then a drill and blast shift of 6 hours each, so completing 2 cycles to achieve 2×3 m rounds in 24 hours.

Constraints to be considered and catered for when contemplating rig introduction.

1. High front end monetary outlays for the rig and initial spares purchase.
2. The kibble winder and openings through the stage must be able to cope with rig mass and dimensions in transport mode.
3. Maintenance and operating crews will have to be more highly skilled than in the the case of conventional operations.

Explosives

Aluminium based water gel explosives and high frequency electro-magnetically initiated detonators are now being employed in some shafts and show promise in:

1. Improved environmental conditions because of their low yield of nitrous fumes.
2. Possible improved safety due to better shock sensitivity characteristics and immunity to stray currents.
3. Ease of use of the loose detonators and the simplicity of threading the toroid into the circuit.
4. Initial tests show economic advantages.

Raise bored centre cores

The lashing portion of the cycle is probably the most arduous of the activities entailed in sinking. Over the last two decades the use of raise bored centre cores, which almost eliminate lashing, has found increasing application. Points which warrant consideration when deciding on the possible use of a centre core are:

1. Bottom access is a prerequisite.
2. The mine's waste handling capability must be able to cope with the quantity of rock which will be generated by the shaft sliping.
3. It is unlikely that the shaft will be an isolated ventilation district and blasting in the shaft may affect the rest of the mine.
4. Points of intermediate access to the shaft site may be necessary to monitor accuracy of the pilot hole and make corrections. They may also be required if the raise drilling system lacks the capacity to cope with the full shaft depth.
5. Shaft spoil should not be permitted to build up in the pilot hole and a chamber large enough to receive an entire shaft round should be cut at the lashing elevations. This action will eliminate a probable cause of hangups in the hole and the danger of mud rushes due to drilling water draining into a full hole.
6. A man-safe grizzley will be required to make the core hole safe for men working on the bottom.
7. Raise boring is costly. This should be assessed against convenience, speed of construction, whether the core can be used over the whole shaft, or if conventional cleaning must be resorted to in the lower reaches of the shaft.
8. Risk of deviation of pilot holes. Precision drilling is still a developing technology and layouts should be designed to accommodate some degree of deflection.

Cover drilling

Most shafts are sunk within long hole cover. Rounds are designed to reduce the likelihood of missing a water bearing fissure. A typical cover round would comprise 36 m deep holes drilled at 85 degrees below the horizontal and so spaced and spun that the toe of one hole would overlap the collar of the succeeding hole on the pitch circle circumference. These rounds would be repeated every 30m so that the shaft would always be at least 6m inside cover.

Heavy jackhammers suspended from below the stage with hemp rope and snatch blocks, 3 m long, circular section extension steels, lugged shanks, water boxes and button or cruciform bits are used. The operator stands on a ladder alongside the machine and controls and assists with the gravity feed. Rods are pulled with the hemp rope usually assisted by a compressed air powered winch mounted on a swivel base in the bottom. All water intersections above a pre-determined quantity are usually injected with a cement/water grout and all holes are pluggea with cement on completion. In areas where heavily water bearing measures are anticipated injections can be carried out from a surface pump shed through in-shaft ranges. In such instances pre-cementation of the shaft site may be undertaken if time permits and circumstances warrant. In some cases pumps are taken to the bottom from where grout mixing and injections take place.

In preparation for a cover the bottom is blown over and the hole positions marked off. Spin direction is laid off by siting from one hole to the next and inclination set with a clino rule. Holes for the casings are drilled with a large bit and the assistance of a centralizing guide dolly. The casings are then grouted, driven in, allowed to take initial set, fitted with drilling cocks and the restraining box girders wedge bolted into position. When the casings have set firm the cover holes are started and drilled about 300 mm into new ground. The casings are then pressure tested with water from a cementation pump and if no leaks are found cover drilling can proceed.

The shaft drill rig is most effective for this type of drilling and cover rounds are executed in about half the time conventionally required. Longer lifts of say 48m are feasible. Very high pressure water intersections could be problematic if the string jammed in the hole so that the drilling cock could not be closed to bring the intersection under control.

Lining

Shaft linings serve several purposes:

To provide local passive support of the shaft wall.

To seal the shaft walls and prevent weathering.

To produce a smooth surface of low aerodynamic resistance.

To provide a precisely located surface from and on which shaft steelwork and service pipes and cables can be accurately and easily installed.

To serve as a foundation for "cast-in" support members, bunton pockets, cast in plates, bearer sets, nut boxes, pipe and cable brackets.

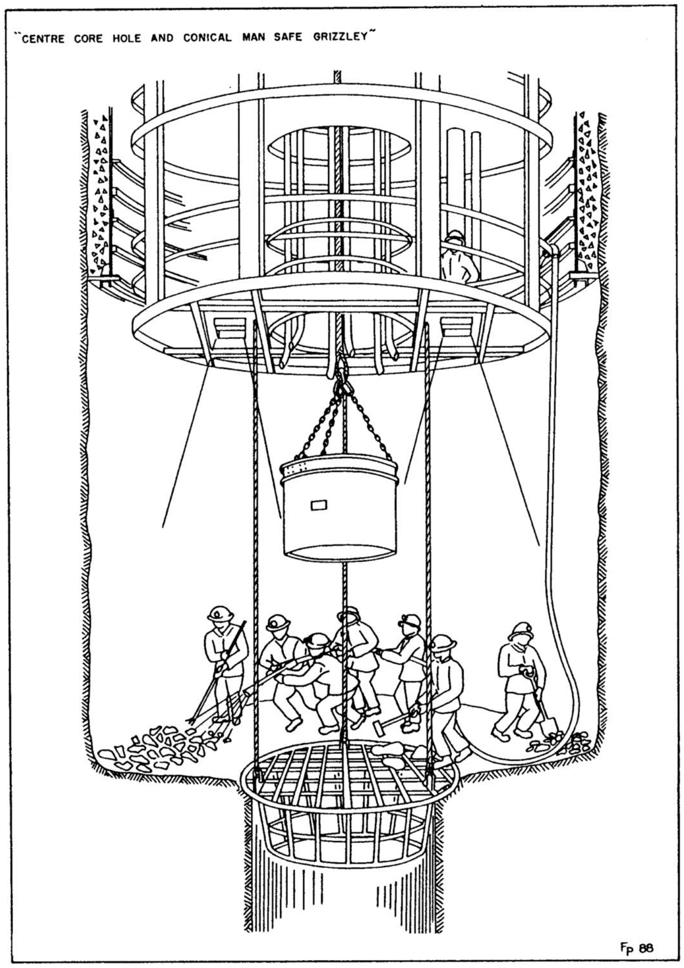

FIG 5 USE OF RAISE BORED CENTRE CORE

Shafts are usually completely lined with unreinforced concrete of minimum 28 day strength of 20 MPa and an 8 hour self supporting strength of 5 MPa for early shutter stripping. Minus 19mm aggregate is used in the mix. In rare instances for reasons of economy just the bunton and pipe supporting curb rings are installed.

The concrete is usually batched and mixed in a plant adjacent to the shaft and dropped down a 150mm diameter steel pipe to a break fall/ remix kettle situated above the top deck of the stage. From thence it is delivered behind the shutter via heavy duty rubber hoses. The manufacture and installation of the concrete delivery columns is of paramount importance to the success of the lining operation. The pipes are heavy walled and manufactured to fine tolerances of length, straightness, flange normality, bolt hole alignment and diameter. Usually 2 columns, one working and one spare, are installed in a precision made bracket. Exact verticality and alignment is achieved by installing with reference to a template and two dedicated plumb wires. Perfect plumbness will virtually guarantee minimal pipe wear. The lining lift height should correspond to the interval between sets in equipped shafts and match the tween deck spacing on the stage. With plain barrels e.g. ventilation shafts it may be convenient to match it with the daily call. This means that the curb ring will be set and the lift poured daily and on the same shifts. If the daily advance of the shaft exceeds the lift length, two lifts will have to poured on some days. The lashing unit requires about 14 metres clearance to operate. This distance governs the timing of curb ring setting. The correct orientation, plumbing and elevation of the ring is of crucial importance and warrants the allocation of a dedicated specialist timberman "Ring man" and gang to the task. Other stage work such as pipe extensions shutter preparation etc. are allocated between the stage hands and gangs of the various shifts so that task responsibility is fixed.

The curb ring usually has the bunton pockets or cast-in chair plate backers, nut boxes etc. incorporated into its design. Its precise positioning will therefore determine the success of subsequent activities such as sinking pipe installations and equipping the shaft with the permanent steelwork.

The shift preceding the "Ring shift" does the preparatory work. Scribing planks are pulled, sorted and salvaged, bunton boxes are withdrawn, nut boxes stripped and most of the ring/barrel flange and key plate bolts are removed, the shutter cleaned and readied for lowering and the concrete pipes extended.

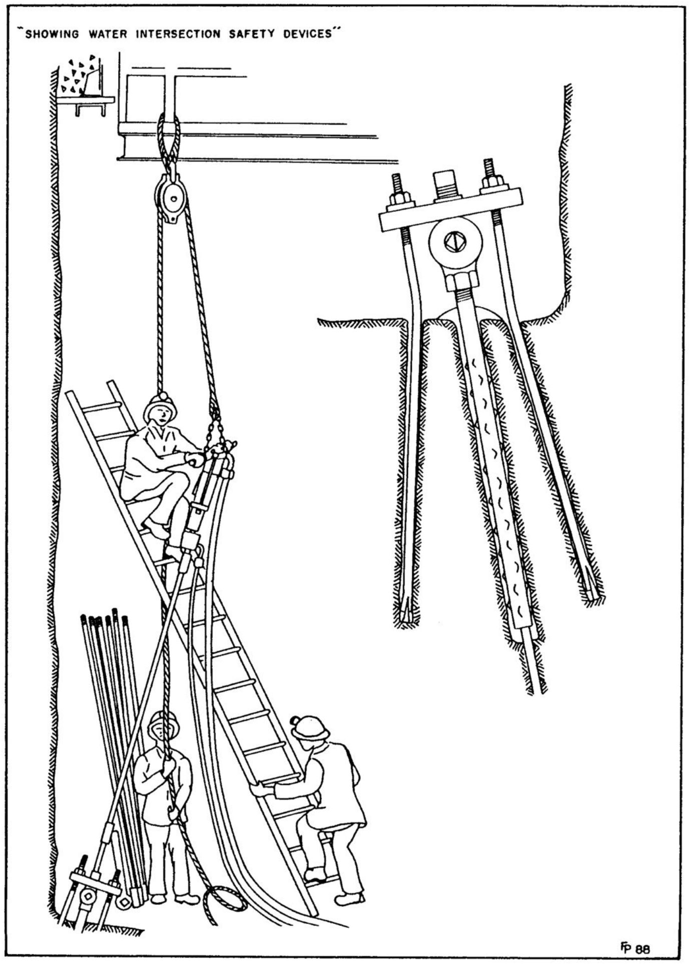

FIG 6 LONG HOLE COVER DRILLING

At the begining of the "Ring shift" after watering and dressing down the shaft walls the shutter winch ropes or chains are attached to the ring and made taut, and the last few bolts removed from the key plate and the flange on the top of the ring. The curb ring is cracked at the key plate and broken free, its weight is taken by the shutter winches and the stage. The stage is then lowered until the ring is at its approximate new elevation. The ring mass is now transferred to the shutter support chains. The stage is locked in this position, jacks extended and lashing can commence. Plumb bobs and steel shaft tapes are lowered to the required elevation and the kettle is attached to the concrete pipe in use. The ring is elevated relative to the shaft tapes with the turnbuckles on the shutter chains. Tape brackets are installed about every 50m down the shaft. It is then orientated by lining up reference marks on the ring with plumb wires with the aid of tangential turnbuckles which have been attached to eye bolts anchored into the shaft wall. It is centred with sprags using offsets from the plumb wires. The number of plumb wires is discretionary, usually about six. It must be borne in mind that it is only in the vicinity of these wires that the ring is true, so they should be sited close to cast in items whose position is critical, such as bunton pockets or chair support plates. Plumb wire steady brackets are extended at about 120m intervals. Scribing planks are inserted, the scribing channel pulled up and the "stop end" sealed with paper or plastic membrane. The curb ring is then poured. Accelerators are usually used in this concrete.

By this stage the shutter winch painters are attached to the barrel plates. The top of these plates is aligned with the concrete of the previous curb. The shutter is now supported from the filler ring which adheres to the lining by skin friction. Once the curb has taken its initial set, the barrel is lowered, aligned and poured. The following shift will check the concrete through the inspection plate and if set, break, lower, align and cast the filler plate and clean off and plaster the newly exposed matching joint. The next shift will prepare the shuttering for lowering and so on.

STATION CONSTRUCTION AND INITIAL DEVELOPMENT

Current practice is to establish the mining levels as and when the shaft bottom reaches the station floor elevation. Development of the station and tip crosscuts and in many cases the raise boring of rock passes and vent raises is executed

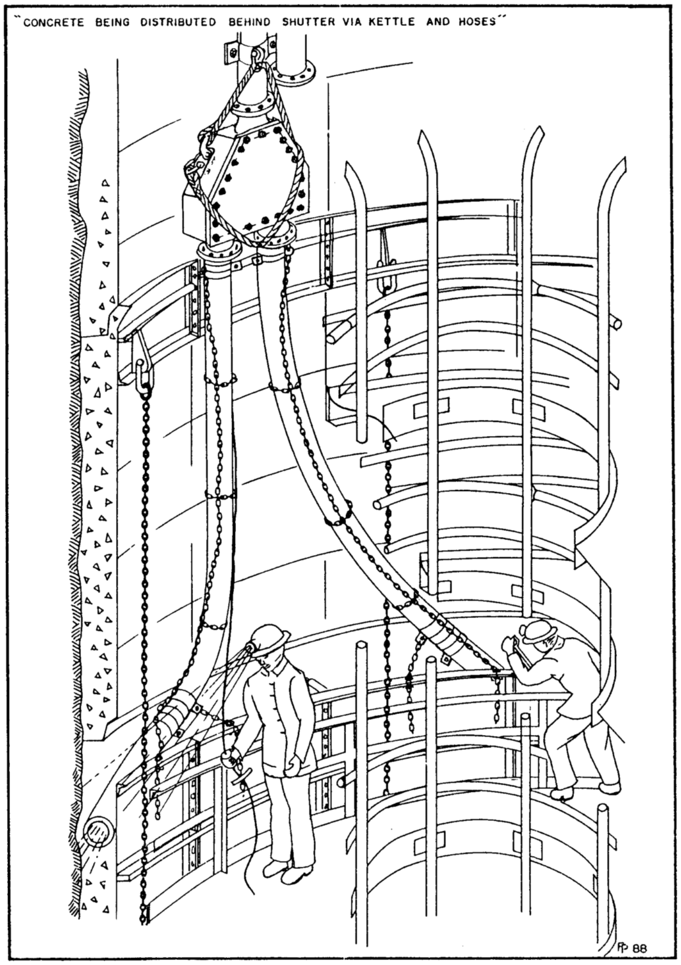

FIG 7 CONCRETE LINING

during the sinking phase. This permits the immediate commencement of development operations and when equipping is complete the shaft system can serve as the relatively well established nucleus of a working mine.

The first cut of the station entries is drilled from the shaft bottom as it advances through the excavation. The shaft is usually taken about two rounds below the required finished floor of the station. This sump is allowed to fill up with muck which serves as a working floor while excavation on the level is in progress. The shaft lining is brought down to the station brow elevation. Depending on personal preference and ground conditions the station can be completely excavated, made safe and supported at this time or the excavation can be limited to that required to permit access for subsequent development.

Crawler-mounted rocker shovels are used to assist with cleaning. They back lash into the shaft area where the grab re-handles the muck into kibbles for final disposal. These methods are applied until a safe parking area for Load Haul Dump vehicles is created. These are usually 1,5 to 2 cu.m machines which can pass through a stage kibble opening and then replace the rockershovels for development cleaning. They likewise feed the grab in the shaft area. These methods yield high advance rates. When levels are to be connected by raise boring, priority is given to the shortest routes to the reamer hook up sites. Rock passes would have been pre-piloted from the upper levels to which access would be gained via drawbridges. Development would be carried on concurrently with raise boring to effectively utilize the available labour and also to generate blasted rock to mix with and dilute the raise borer chips which the grab has difficulty handling.

In instances where it is considered advantageous to carry out extensive development concurrently with shaft sinking, mid shaft loading arrangements can be considered. In these cases a section of the shaft would be equipped for skip hoisting from a loading box so that sinking and development operations could proceed independently of each other.

On completion of development the station excavation is opened up and/or trimmed to size. The shaft is lashed out and the curb ring set on the station floor and poured. This then forms a floor from which the station formwork and central barrel can be built. Progressive piecemeal pours can be made, or the station shutter can be completely erected, stop ended off at the concrete limits and married into the shaft plates at the brow/shaft intercept and cast in one continuous pour.

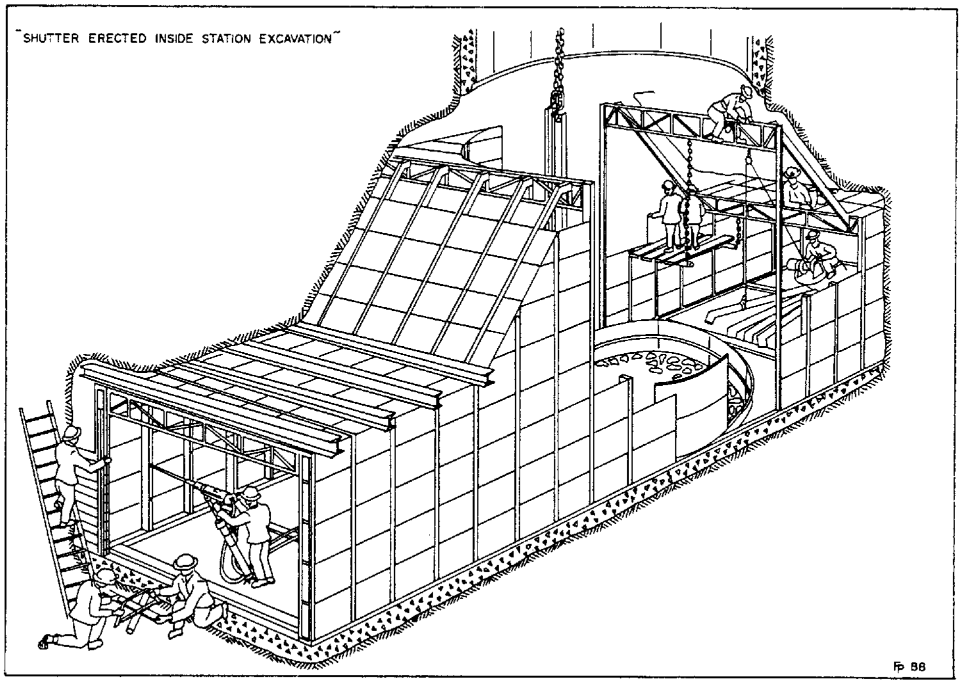

FIG 8 STATION CONSTRUCTION

Loading pockets are generally formed by creating a large chase on one side of the shaft in the floor of the belt station. The chase is usually carried as a bench above the bottom and lining is done in sections. When a lift has been excavated the curb is set in the shaft, tied back and poured together with a construction sill cast on levelled off dirt in the chase. The shutter is then built off this base and married into the shaft barrel plates. This process is repeated as required until the pocket is complete.

The final activities of this phase of operations are to take the lining into the bottom and to blow over, examine the bottom certify it as clear of misfires and then pour a sealing floor screed over it.

EQUIPPING

Steelwork jigging, checking and guide drilling

Shaft steelwork and guide blanks are ordered well before they are required for installation so that delivery is effected while adequate time is still available for jigging, checking and guide drilling.

Before being marked, worked or installed all steel is stored out of the sun under a roof for at least 12 hours.

Shaft steelwork is usually corrosion protected. Epoxy coating was once almost universal but galvanizing is now gaining in popularity. Indeed in some recent instances not only has the shaft set and station steelwork been galvanized but the guides as well. Fastenings are treated with various patented processes.

Bunton sets are almost invariably of aerofoil or flattened pipe section. They are jig fabricated and assembled at the manufacturers, checked, passed by the purchaser and sent to site. Here they are reassembled in a similar jig, numbered and reference marked for plan position and elevation. The sets are bolted up to precisely positioned guide stubs and the buntons are pop marked with centre punches on the tops and sides of purpose fitted target plates. Offsets from these marks to the plumb wires and tapes will be used to locate the set in the shaft on installation. A sketch of each set is marked up with any irregularities found and what remedial action was taken. On dismantling each set its members are marked and stored as a complete unit until moved to the shaft for installation.

Headgear steelwork to be installed during the changeovers from sinking to equipping and finally to permanent condition is check assembled. This is also the time to trial erect station and loading box steelwork.

Guide cutting drilling and matching is a factory type production line activity. Modern rolling mill practice with continuously adjusted rolls produces guides to very fine tolerances which virtually eliminates difficulties in matching.

Guides are cut to dead lengths and their ends squared up. They are then clamped in the drilling jig and drilled through fold over templates. They then move down the line and are successively countersunk, notched for the nibs on the bolt heads, butt straps are attached, matched to the preceeding guide, numbered, colour coded and finally stacked in runs and stored until installed.

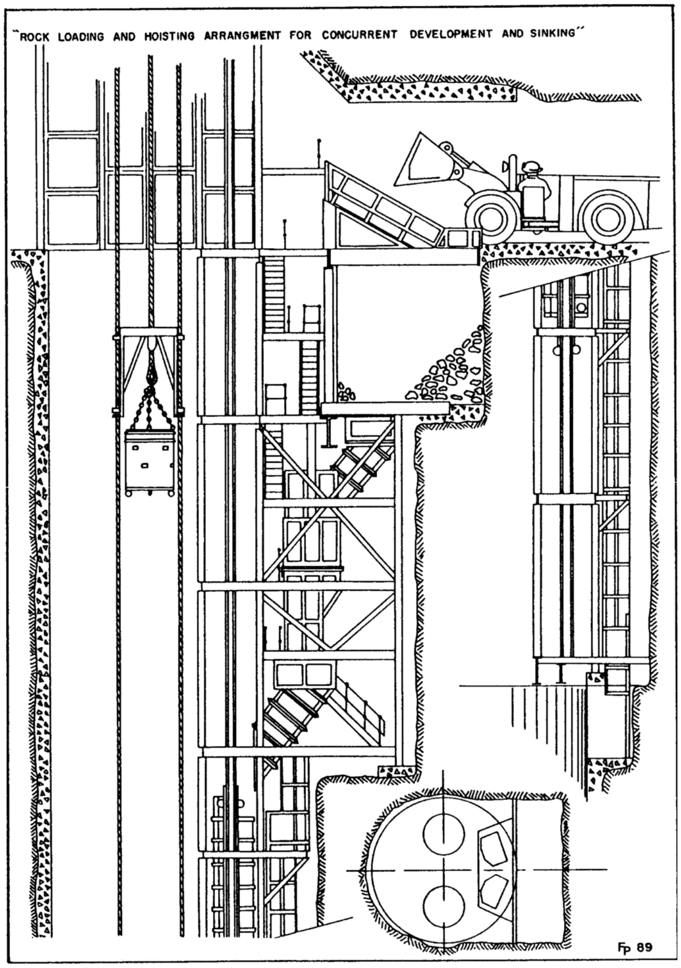

FIG 9 MID SHAFT LOADING

The up run

Stripping and pipe installation

On completion of sinking the shaft bottom is cleaned and the lashing unit is stripped out and sent to surface. This is followed by the shaft shutter. Large pre-assembled sections of the loading box can be lowered at this juncture and erected in the pocket.

The permanent shaft pipes are frequently installed during stripping. The stage is progressively raised to surface and the shaft stripped of all sinking services pipes etc. The bank doors are removed and use is usually made of the stage to strip the temporary steelwork from the headgear and change it over into the equipping condition.

Brattice walling

The concept of brattice walling combines upcast and downcast ventilation facilities in the same shaft.

If a brattice wall is to be installed this will require stage modifications to split it into trouser legs. The wall would be built from the bottom up.

The walls consist of precast, prestressed, reinforced, tongue and groove concrete panels of about 1m high×0,3m wide×shaft diameter+ about 0,30m for the wall sections×shaft diameter+about 0,45m for the support panels. A chase to accommodate the brattice wall is formed in the shaft lining on opposite sides of the shaft. A deeper recess is formed over the curb ring height to take the support panel. This member is placed in these recesses groove uppermost, it is levelled and elevated with packers, centred and chase grout plates bolted up against it on either side. The groove is then filled with a sealant, usually an epoxy resin. A wall panel is then placed on top of the support member tongue down, the panel positioned, grout plates put in place and this process repeated for another 3 or 4 panels after which the lift is grouted up and another support panel installed. The whole process is repeated up the shaft to surface where the upcast section is capped with a transition curve into ducts leading to the

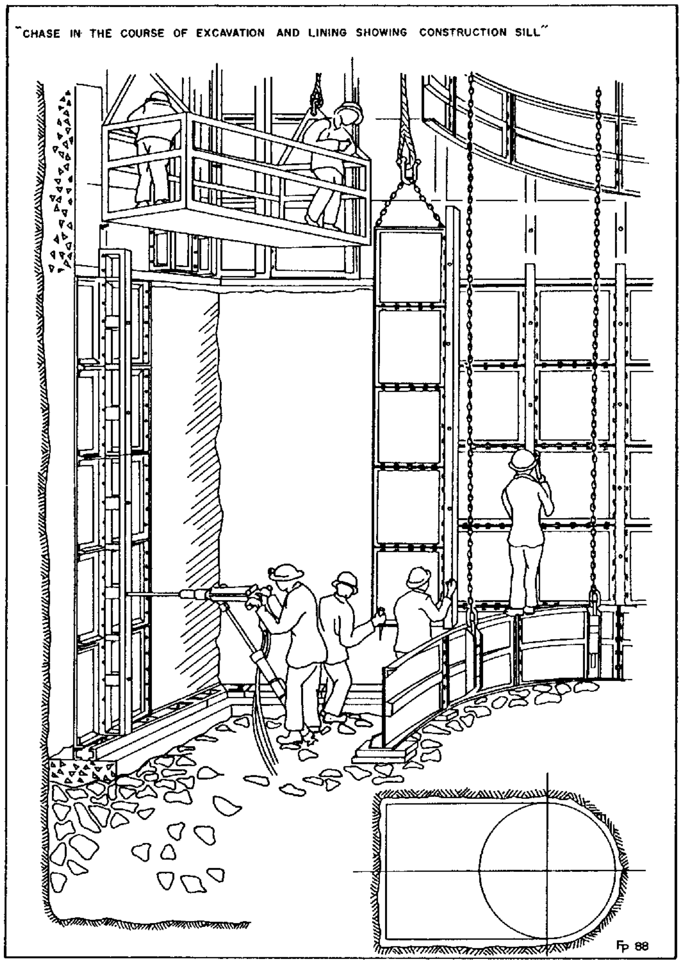

FIG 10 LOADING POCKET CONSTRUCTION

permanent fans. A stage made to suit the downcast area is usually used to install the shaft sets in the down cast section from the top downwards.

Shaft equipping

This is the climax of probably two to three years work.

The benefits of precise workmanship during sinking and lining will now be harvested or the penalty paid for past omissions. Equipping conveyances are now installed and commissioned, the stage modified, the bank steelwork surveyed in and grouted, steady and tape bracket positions established. A variation which has been used with success is to install steady brackets up to 120m below the steelwork. Set positioning is then effected from static wires and the lining up process expedited and accuracy of installation improved.

Equipping invariably takes place from the bank down with the stage locked at a convenient working height below each set installation position.

Cut outs are frequently provided in the stage decks at the guide positions. Guides can then be stowed vertically in the stage and project out of the top deck at a suitable height to facilitate the splice with the upper guide.

If this method is not employed then the guides are installed from skeleton cages above the stage. If chair plates are being used they will be installed from the second deck, otherwise bunton pocket clearing will be carried out. The shaft set components are slung below the skeletons and lowered to the stage in the compartment best suited. Members are received on and moved into position from the top deck and the set loosely assembled. It is then lined up, elevated and bolted. Before it is finally tack welded or grouted it will be examined, check measured and passed by a surveyor one of whom should be available at all times solely for this purpose. The relevant checked measurements are marked on pre-printed pro-forma sketches which are completed for each set, signed for and kept on file as a section of the shaft's history.

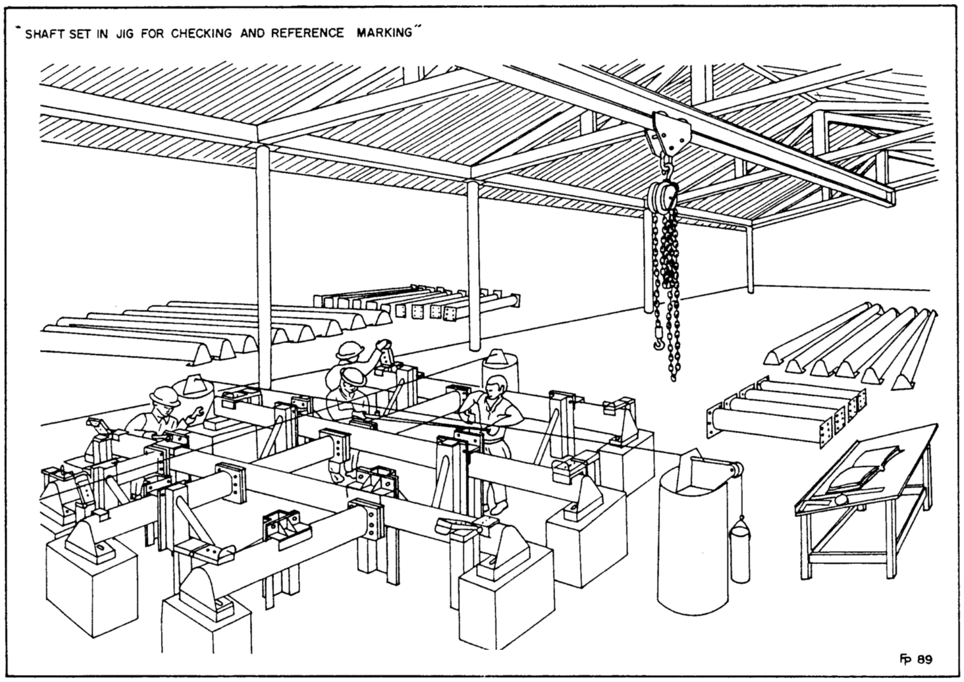

FIG 11 STEELWORK JIGGING

Several types of self locking fastenings for steelwork assembly are in use. Amongst the most popular is a hydraulically tensioned pin with a crimped on collar. Stations are equipped as and when they are encountered.

Frequently shaft barrel equipping is only interrupted for the installation of the main structural members. An additional crew then completes the equipping of the station concurrently with the barrel. When equipping reaches the shaft bottom, the accumulated debris is cleaned out, the stage cut up and sent to surface and the stage ropes pulled out of the shaft. The final headgear changeover from equipping to permanent condition is effected, shaft cables and the permanent conveyances installed. The station tracks are now lined up with those of the conveyances and concreted in, after which the whole system is commissioned as an operating entity.

Considerable time and effort is dedicated to the achievement of high standards of accuracy in equipping. Generally accepted installation tolerances for steelwork are +−3 mm on plan and elevation. This ensures the very even winding conditions achieved at speeds of up to 18 m/sec.

CONCLUDING COMMENTS

Local deep level mining operations are conducted on a scale unmatched elsewhere. The environment, depth and magnitude of the operations which are required to coax the earth to yield its treasures have demanded the development of techniques, tools and traditions which are unique to the South African Mining Industry.

This is also true of our shaft construction industry which establishes access ways to these workings and sets standards of occupational safety, product excellence and production rates that are unequalled elsewere.

GLOSSARY

BANK the reference plane from which elevations are measured. Either above or below bank.

BANK DOORS in sinking condition the shaft is completely decked over at bank elevation with a substantial safety bulkhead. Openings are required in this bulkhead to permit the travel of conveyances through the shaft and headgear. These openings are normally fitted with pneumatically powered hinged doors (bank doors) which are closed when:- kibbles are being tipped, detached from or attached to the ropes and men or materials are being loaded or unloaded.

BARREL PLATES the long section of shaft shutter plates between the curb and the filler rings. This section of plates is usually handled as a unit, it is broken out lowered and attached to the already poured curb ring. The top outer edge of barrel plates is then aligned with the curb ring concrete of the previously poured lift.

BRATTICE WALL a substantial structure of prestressed reinforced concrete slabs, let into chased slots in the shaft wall and grouted into place. The purpose of which is to create upcast and downcast ventilation sections within the same shaft.

BUNTON a main horizontal member of a shaft set, both ends of which are attached to either the shaft wall or another bunton and the shaft wall. The main purpose of which is to support the guides.

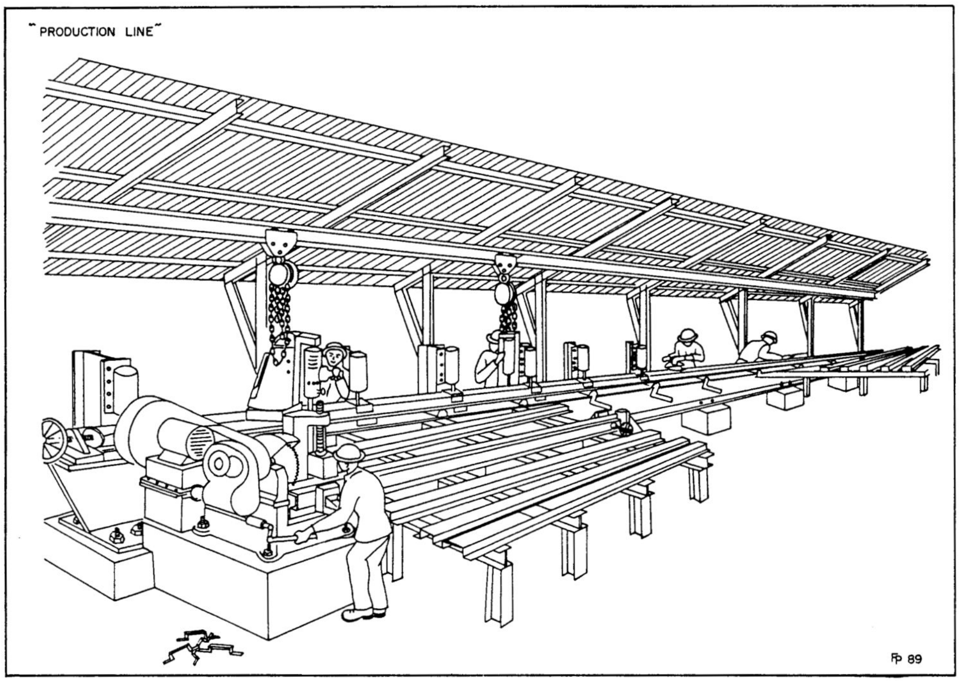

FIG 12 GUIDE CUTTING, DRILLING AND MATCHING

CACTUS GRAB the digging and loading device used with the lashing unit for shaft cleaning. It consits of a large pneumatic cylinder which powers a cluster of sharp pointed inward curving tines. These open and close and dig their way into the muck pile. When full the tines are closed around the load of rock, the grab is then raised, positioned above a kibble and the tines opened so releasing the load and dumping it into the kibble.

CHAIR PLATE a support attachment bracket for bunton ends, it is bolted onto a foundation plate cast into the shaft lining. The method is gaining in popularity, it offers the following improvements over the traditional grouted pocket methods:- installation is faster, neater and stronger, no mess from grout spillage and damaged buntons are easier to replace.

CONSTRUCTION SILL this forms a base from which to build formwork in excavations which are cut and lined from the top down such as in loading pockets and in instances where ground conditions do not permit complete excavation and lining. After the lining has been poured and set and the shutter stripped, the sill is broken out and the process repeated with the next lift below.

COVER DRILLING this is the process by which the ground to be traversed below the bottom or behind the face is probed for the presence of water or gas prior to its treatment by injection.

CROPPERS these are the perimeter shot holes in a shaft bottom round, they are usually lightly charged to minimize blast damage to the shaft wall rock.

CURB RING this is the bottom section of the shaft shutter and lining lift. Its successful installation is critical to the effectiveness of the lining and subsequent equipping operations. It is stop ended off and forms the base from which the rest of the lift is built. It carries most of the cast in brackets, nut boxes and bunton pockets. The subsequent lift is matched up to it.

CUT the first portion of the round to detonate and create a breaking face for the rest of the shot holes and so facilitate face advance.

BURN CUT a cut in which a cluster of closely spaced parallel holes are drilled around a central uncharged hole termed a stab hole.

CONE CUT a cut in which a ring of holes started on a large diameter pitch circle on the bottom are drilled towards each other to terminate on a much smaller diameter concentric pitch circle, the round length below the face.

DIVIDER a smaller section horizontal member of the shaft set either between 2 buntons or the shaft wall and a bunton. Their main purpose is to stiffen the buntons.

FILLER RING the top section of shaft shutter plates. It is from this ring that the curb ring and barrel plates are suspended while the concrete behind them is setting. The filler ring is then attached to the barrel plates and matched up to the curb concrete of the previous lift. Concrete is then poured behind this shutter to complete the lift.

INERT WAX PRIMER this is the durable hard wax cylinder in which the long period delay detonator is inverted and encased. This is always placed in the bottom of the hole, with the lead wires coming from the top of the primer. Its use reduces the danger from accidentally detonated misfires because it ensures that the bottom of the misfired hole is largely filled with non explosive material with the exception of the detonator itself.

KETTLE a device used on the lower end of the concrete column, it breaks the fall of the concrete and ensures its remixing just before it is discharged behind the shutter.

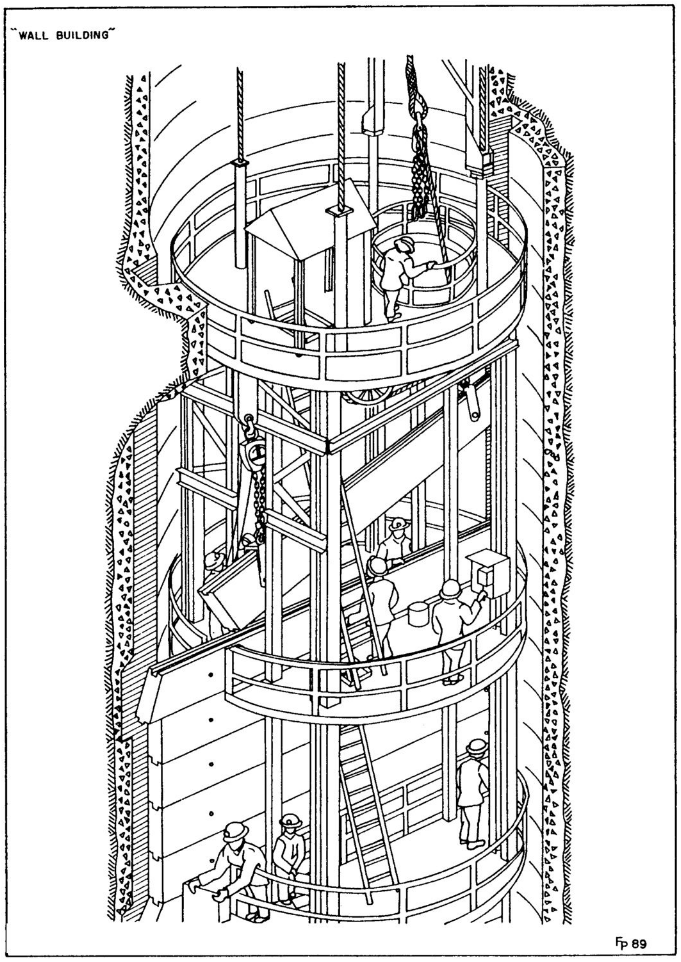

FIG 13 BRATTICE WALLING

KEY PLATE a section of plate in the shaft shutter whose vertical flanges are tapered together towards the outside of the plates. The shutter is broken at the key plate in the stripping process.

KIBBLE a large cylindrical bucket supported in detachable chains which forms the main means of conveyance for men, equipment and material in the shaft and for the removal of rock and water.

LASHING the loading of blasted rock into conveyances for its disposal. Hence LASHER one who lashes and LASHING UNIT a mechanical device incorporating hoisting, slewing and radial traversing mechanisms for the handling of the cactus grab which completes the lashing system.

LOADING POCKET a large chase cut in the side of the shaft to accommodate the rock handling and skip loading arrangements.

MACHINE the term for a jackhammer. Hence.

MACHINE KIBBLE a short kibble for ease of handling when packing and unpacking rockdrills, with a central cylindrical quiver to hold the drill steel and below which the compressed air and water manifolds and hoses are slung for quick and easy transport into and out of the shaft and bottom.

OMEGA CLIP a fairly stiff plastic tube incorporating a single longitudinal slit, of similar cross section to the Greek letter Omega. It is used for making up and holding together the light and often decoupled charges used in the croppers.

PRE-CEMENTATION the treatment of long sections of the shaft site with deep holes and the cementation process prior to the commencement of sinking. The main advantage of this technique is that the injections are carried out by small specialist crews, so the time so spent, is not as expensive as that involving full shaft crew and plant stoppage.

RING MAN a specialist shaft timberman dedicated to the task of setting the curb ring.

SCOTCH CRANE a simple stationary hoisting arrangement sited on the bank along side the shaft and used for pre-sinking. It incorporates a winch, a swiveling upright derrick and a slewing jib.

SCRIBING (1) the marking off and stub drilling of the cropper holes on the shaft bottom. (2) the stop ending off of the curb ring and indeed of any shutter.

SINKING STAGE this is the mobile multi deck platform which serves many purposes:- it forms a protective cover for the shaft bottom, a scaffold from which to set shutters and pour linings, a bottom anchor for the stage support ropes which form the kibble crosshead guides, second means of egress, distribution base for power, water, compressed air and concreting, a

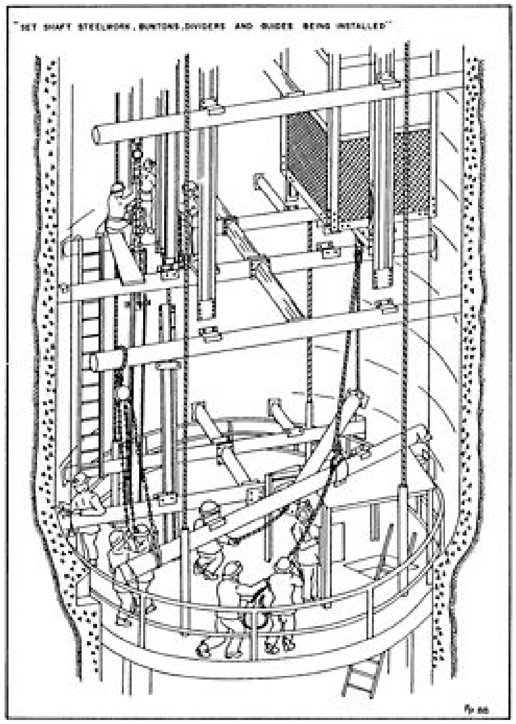

FIG 14 SHAFT EQUIPPING

base station for signalling and from which to illuminate the working places, a storage area for material and equipment and a base on which to mount the lashing unit.

SKELETON CAGE a mobile framework of platforms which runs on the permanent guides and is used for the transport of men, material and furnishings during equipping and to assist with the installation of the permanent furnishings.

SMOOTHEX a trade name of a small diameter cartridge of low intensity explosive used in the peripheral holes of excavations such as croppers. The cartridges incorporate spigot and socket ends for joining them together to make up a long charge.

SOCKET PLUG a tapered wooden block used in South Africa to insert into the residual portion of a blasted shot hole after it has been cleaned and examined to indicate that it is safe.

STATION this is the horizontal landing cut and constructed out of the shaft from which the levels can be broken away.

TOE-IN this is the widening out of the shaft lining at the base of the collar at the elevation at which the shaft enters fresh rock to form a firm foundation for the shaft lining.

TROUSER LEG this is the splitting of the lower decks of the stage into sections so that they can fit into the shaft compartments. The technique is most commonly encountered when brattice walls are being built and the stage is split into 2 sections with a bisecting slot running between the bottom and the underside of the 3rd deck. This configuration allows the grouting up of the brattice wall into the chase between the bottom and the 2nd deck and the building of the brattice wall between the 2nd and 3rd decks. They are also used in some instances when bottom up equipping is undertaken.

Rehabilitation of the Armistice shaft

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ABSTRACT

In Canada, a number of former producing underground gold mines are being re-examined and the shafts rehabilitated. Some of these shafts are also being deepened. Many of the shafts are timber construction and have small compartments thus making access for either modern mining equipment or shaft deepening equipment difficult. Dynatec Mining Limited have devised a procedure for replacing the timber sets with concrete rings and thereby increasing the size of the compartments and the integrity of the shaft. This procedure was first used at the Armistice Resources property near Kirkland Lake, Ontario. The paper describes the entire operation from stripping old timber to pouring concrete rings, installing new guides and preparing to deepen from 1275 feet to 4000 feet.

PAPER;

Armistice Resources Ltd. has optioned 34 claims from Sheldon-Larder Mine Ltd. These claims are an assembly of properties originally held by Armistice Gold Mines Limited (no connection to Armistice Resources), Arjon Gold Mines Ltd. and Sheldon-Larder itself. Initial exploration on this group of claims started in 1937 and over the years dozens of drill holes and trenches have been completed and a number of shafts have been sunk. The project being reported (at this time) is only concerned with one of these shaftsthe vertical Armistice shaft, 1275 feet deep, near the southeast shore of Barber Lake.

The Armistice Resources Limited's property is located in northern Ontario about 450 miles north of Toronto. It is about a half mile west of the village of Virginiatown, Ontario. The Armistice shaft is 700 feet north of Highway 66 which is the main road running between the mining communities of Kirkland Lake, Ontario and Noranda-Rouyn, Quebec. (Fig. 1) The Armistice claims are located along the Larder Lake Break. This is one of two major linear structures in northern Ontario and Quebec. The Larder Lake Break stretches from Val d'Or, Quebec in the east, past Rouyn-Noranda and Kirkland Lake to Mattachewan, Ontario in the west (Fig. 2). A large number of Canada's gold deposits are associated with this structure. One of these, Kerr Addison Mines, is located east of the Armistice property and they share a common boundary.

In the spring of 1945, Armistice Gold Mines Limited obtained permission and financing to sink a 675 foot deep shaft with planned development on the 275, 425, 525 and 650 foot levels. The shaft was a three compartment timber shaft (Fig. 3). By February 1946, the shaft had been sunk 675 feet and the stations and level development completed on the four levels. (Fig. 4)

As a result of this work a total of four zones had been identified above the 650 foot level with reserves of 75,000 tons. However, the results were less than anticipated and in March 1947, Armistice announced plans to deepen the shaft to the 1250 foot elevation.

During sinking, stations were cut at the 800, 950, 1100 and 1250 foot levels. At the 1250 level a cross-cut was developed north into the carbonate zone. This zone was over 200 feet in width. Three flow bands were identified and drifting was carried out on the centre flow band.

The north cross-cut on the 1250 foot level also identified a syenite dike, well mineralized with pyrite, and entered well mineralized black lavas. Drifting along the contact of the lavas and green carbonates returned assay values varying from 0.28 ounces per ton to 0.04 ounces per ton.

With funds diminishing in late 1947, underground development was halted to undertake horizontal drilling of the carbonate zones from three levels. On the 1250 level several promising holes were drilled in the spring of 1948. Four of these holes returned assay values from 0.12 ounces per ton to 0.28 ounces per ton. All of the values were in the silicified flow zone. In October 1948, Armistice halted operations and dismantled their plant.

There were no further operations at Armistice until the mid-1970's at which time Kerr Addison (who had an option on the property) drove an underground drift on their 3850 level to within 800 feet east of the shaft section and 1000 feet north of the anticipated flow zone. Horizontal holes were drilled from this drift and some gold values were intersected. Since the price of gold continued to be unexciting and the drill results not encouraging, Kerr Addison dropped its option and in 1978 title was returned to Sheldon-Larder.

In 1984 Aurelian Developers optioned the property from Sheldon-Larder for property payments and work commitments. In 1986, Aurelian transferred the option to its subsidiary, Armistice Resources Ltd. Armistice has carried out surface diamond drilling and feasibility studies. They identified two possible approaches to the exploration of the deep target area.

1. Rehabilitation of the 3850 foot level drift from Kerr Addison Mine and extension to the Armistice west property boundary with an underground diamond drilling program.
2. Rehabilitation of the old Armistice shaft to the 1250 foot level and deepening of the shaft to the 4100 foot level with an accompanying program of drifting and underground diamond drilling. Under this program a major portion of mine development costs would have been met during exploration.

In 1987, Armistice Resources Ltd. selected deepening of the existing shaft as the best method of exploring the deep target indicated by drilling completed in 1974. The exploration target was the "Kerr Zone". This zone is believed to extend from the 2000 foot level to below the 4000 foot level, and about 1000 feet north of the Armistice Shaft. The main objective of the program was to test the continuity of mineralization between diamond drill hole intersections and classify the gold mineralization as proven and possible reserves.

In September 1987, Dynatec Mining Limited received a Request for Proposal to re-establish access to the Armistice shaft and underground workings, to carry out shaft sinking, exploration drifting and cross cutting and exploratory diamond drilling. The work was to commence in October 1987 and be completed by December 1989 (Fig. 4).

The general outline of the work to be completed was as follows:

Phase 1:

1. Mobilization and installation of surface facilities.
2. Dewatering of the 1275 foot deep, three compartment Armistice Shaft.
3. Rehabilitation of the shaft and four stations including lip pockets.
4. Rehabilitation of the four existing levels, totalling approximately 2000 feet.

Phase 2

1. Deepen the existing three compartment shaft by 2825 feet to a depth of 4100 feet. Overall shaft excavation to be 19 feet by 8 feet.
2. Excavate and equip, complete with station doors and loading chutes, seven stations 40 feet out from the shaft and seven lip pockets on the 2800, 3000, 3200, 3400, 3600, 3800, and 4000 foot horizons.

Phase 3:

1. Excavate 4600 feet of drifting and crosscutting on the 4000 foot level horizon.
2. Complete 20,000 feet of underground exploratory diamond drilling.
3. Extract and transport bulk samples collected from the backs in drifts and crosscuts to surface.

Phase 4:

1. Demobilization and clean-up of site.

The contractor was to furnish all labour, supervision, materials and supplies, equipment and plant.

Following receipt of the request, Dynatec formulated a plan to carry out the work as requested.

During the planning stages of a shaft sinking project, one of the first items considered is the selection of a sinking hoist. This is important for two reasons.

1. If the incorrect hoist is selected the cost and time for shaft sinking can increase dramatically.
2. If the sinking hoist can also meet production requirements there can be considerable savings for the owner as only one hoist is required for both sinking and production.

Selection of a sinking hoist has been discussed by others and the principal consideration has been summarized as follows. (1)

> “For any hoist, the capacity is a function of the payload and speed of the conveyances. An increase in payload spells an increase in capital cost; an increase in speed means an increase in energy consumption and hence, operating cost. Determination of the correct balance between speed and payload may be controversial in the case of a production hoist, but it is relatively straight-forward in the case of a sinking hoist. As will be seen, the portion of sinking cycle period devoted to full speed hoisting is typically less than half the total cycle time required. The result is that it is more efficient to achieve the required capacity through increased payload than it is to do so by increasing the hoist speed. In other words, “the bigger the bucket, the better”.”

At Armistice the existing shaft hoisting compartments were sized at 5 feet by 5 feet with 6 inch by 4 inch guides. The face-to-face distance between guides was 4 feet 4 inches and would accommodate a shaft sinking bucket with a capacity of 2.5 tons.

Preliminary calculations indicated that during sinking there would be 20 buckets per bench and the time to muck out the last bench, with a hoisting speed of 1800 fpm and efficiency of 70%, would be 2 1/2 hours. The number of buckets per bench and mucking out time were both greater than would normally be considered acceptable.

In addition to limiting the sinking bucket, it was felt that the compartment dimensions would also limit the dimensions of the permanent conveyances and have a detrimental affect during the exploration program and long term operation of the shaft. Small conveyances limit the number of men that can be transported at one time. With small compartments much of the mine equipment must be dismantled or cut into smaller segments thus increasing lowering and re-assembly time. In addition, the smaller compartments result in a higher ventilation resistance factor.

After making a preliminary estimate for deepening the shaft as requested, and confirming the impact of the smaller compartments, the Dynatec contracting and engineering staff came together to “brainstorm” a solution to solve all of the above problems. They knew that the solution would have to be practical, economical and acceptable to Armistice. It should be remembered that at this stage Dynatec were in a competitive bidding situation and there was a large project at stake!

The focus of attention was on finding some way of increasing the existing shaft compartments to 6 feet by 6 feet (inside timber) , and sinking the remaining shaft with 6 feet by 6 feet compartments. The reason for this is that in Canada many shafts have been designed with 6 feet by 6 feet compartments and, as a result, much of the existing equipment has been designed to be broken down to fit these dimensions. There are also used cages, skips, hoists, etc., that have been designed to suit these dimensions.

After reviewing the shaft layouts provided by Armistice, it was felt that if the timber set and posts could be removed, and the space between the outside of the timber and the wall rock filled with concrete the remaining opening could accommodate two hoisting compartments 6 feet by 6 feet 4 inches plus a 4 feet 6 inches by 6 feet 4 inches manway compartment.

A plan view of a typical set of the timber shaft and the proposed concrete ring shaft is illustrated in Fig. 3. The concrete is a 2 feet high ring reinforced with 3/4 inch rebar on 12 inch centers. The guides have been increased to 6 inches by 8 inches to permit the use of larger conveyances. The guides are supported by brackets bolted to the concrete on one end wall and the 8 inch by 8 inch timber dividers. These dividers are supported in cutouts on the top of each ring. The manway landings are supported by the divider and the concrete.

The manway has screening between it and the hoisting compartments. Pipes have been located in the manway and are supported on brackets embedded in the concrete. The concrete rings were to be poured on 6-foot centers with the base of the ring at the same elevation as the top of the former timber sets.

Once Dynatec were satisfied that this new configuration would meet Armistice requirements and after devising a procedure for installing the concrete rings, they had to show that the extra cost, and time, associated with upgrading the existing shaft, could be justified. This was done as follows.

1. **<u>SAVINGS IN SHAFT DEEPENING</u>**

   Hoisting calculations indicated that during shaft sinking the 6 feet by 6 feet–4 inches compartment could accommodate a 5 3/4 ton bucket as opposed to the 2 1/2 ton bucket; that the number of buckets per bench decreased from 20 to 11; and mucking out time, (for the last bench) decreased from 2 1/2 hours to 1 1/2 hours. In total, it was felt that the sinking rate would increase from 7.9 feet per day, for the 5 feet by 5 feet compartment shaft, to 9 feet per day, for the 6 feet by 6 feet-4 inches compartment shaft. This meant that the total time for the 2875 feet of sinking would decrease by 43 days. The daily cost of a sinking plant including labour and equipment is approximately $15,000 and the total direct savings would be $645,000.
2. **<u>SAVINGS IN HEADFRAME HEIGHT</u>**

   Hoisting calculations also indicated that the “ideal” hoist for both sinking and future requirements would be 10 feet in diameter with a 6 foot wide face. Using typical hoist and hoist rope geometry it was shown that the headframe for the 6 feet by 6 feet-4 inch compartment shaft, with hoisting ropes at a vertical angle of 45 degrees, would be 15 feet shorter in height than for the 5 feet by 5 feet compartment shaft. Headframe of this type typically cost $6,000–$7,000 per foot of height.

3. **INCREASED PRODUCTIVITY**
   The floor area of the cage in the 6 feet by 6 feet-4 inches compartment shaft is approximately 28.6 square feet and could accommodate 16 men per deck whereas the floor area of the 5 feet by 5 feet compartment shaft is 18.4 square feet and can accommodate 9 men. This is an increase of 78%.
4. **VENTILATION**
   The area of the shaft excavation, minus area of timber, manway, concrete etc., for the two shafts was compared to determine a net available area. For the timber shaft the net available area was 49.3 square feet whereas for the concrete ring shaft the net available area was 80.7 square feet. A difference of 64%.
5. **EQUIPMENT**
   As mentioned previously, the lowering of equipment is often more difficult with smaller shaft compartments.
6. **SHAFT REHABILITATION**
   During the bidding stage, when the economic analysis was being completed, the shaft collar was capped with a reinforced concrete bulkhead. A small diameter hole had been drilled through the bulkhead to determine the elevation at the top of the water. However, it was not possible to ascertain the condition of the shaft walls or the timber. Experience with similar shaft rehabilitation projects indicated that the amount of rehabilitation could vary from nil to 100% timber replacement and rock bolting of walls.

For the purpose of comparison, Dynatec made the following assumptions:

1. Timber Shaft

Assume shaft could be repaired by dewatering crew. Assume 50% of timber and 100% of hanging rods,

2. Concrete Rings

Assume that crew and equipment would be larger than required for dewatering only and that 100% of timber would be stripped and replaced with concrete rings. All pipe, brackets, valves, ventilation ducts would be new. The cost of this work (labour and materials) was estimated to be $1,014,000.

Table 1 summarizes the differences in cost, and hoisting capacity between the 5 feet by 5 feet compartment timber shaft and the 6 feet by 6 feet compartment concrete ring shaft.

As can be seen, the difference in estimated cost between the two shafts was small (less than 1%). It was Dynatec's opinion that the long-term advantages to Armistice would far outweigh the difference in costs.

A responsive bid package, which also included Dynatec's alternative, was sent to Armistice and, after some discussion, the project was awarded to Dynatec in October, 1987.

In order to complete the project in a timely manner, Dynatec decided to divide the work into two phases. Phase 1 consisted of preparing the site, refurbishing the collar, establishing power, water and other services, detwatering and rehabilitating the existing shaft. This work was to be completed using a temporary headframe and hoisting plant. Phase 2 consisted of deepening the shaft from 1275 to 4100. This was to be completed using a larger hoist and headframe that would also meet the production specifications mentioned previously. This plant would be built at the same time as the Phase 1 shaft refurbishing work was in progress.

The Phase 1 sinking plant consisted of a steel, fold-a-way headframe 32 feet high sitting directly over the shaft collar. A skid mounted single drum hoist 45 inches in diameter by 30 inches wide was installed approximately 45 feet south of the shaft. The hoist was roped with a 3/4 inch diameter hoisting cable and was capable of hoisting a 8,000 pound load at a speed of 200 feet per minute. A 65 horsepower electric motor provided power to the hoist.

Electric power was supplied by one diesel generator.

Construction work started October 16, 1987 and consisted of levelling the site, removing the old collar, constructing a new collar, installing collar doors, erecting the portable headframe and hoist, setting up the diesel generator and establishing an office, change house and shop. This work was completed January 11, 1988.

In order to dewater the shaft four types of pumps were used. These were Flygt BS-2125, 2 stage, 13 HP, Flygt BS-2201, high head, 58 HP, Flygt BS-2400, high head, 140 HP and a CIR 2GT, 75 HP. These pumps have pumping capacities of 250 to 1,000

TABLE 1

Comparison Between Timber Shaft and Concrete Ring Shaft

Item	Timber Shaft	Concrete Ring Shaft
Mobilize & Set-up	1,090,000	1,090,000
Dewater	357,000	357,000
Rehabilitation	225,000	1,014,000
Deepen Shaft	4,903,000	4,903,000
Shaft Deepening Savings	–	(645,000)
Common Costs	5,316,000	5,316,000
Headframe Savings	–	(100,000)
TOTAL:	11,891,000	11,935,000
Average Sinking Rate 7.9 ft/day9.0 ft/day		
No. of Men/Deck	9	16
Ventilation Area	49.3 ft^2	80.7 ft^2

gpm depending on discharge head. A 6 inch steel pipe was installed in the shaft manway as the permanent discharge line. The pumps were connected to a 4 inch pipe line and/or hose which fed into the 6 inch line. At various stages compressed air was used to “boost” the pumping capacity of the various pumps.

Initial pumping was started with the Flygt BS-2125 submersible pump. It was suspended from a 1/2 inch cable attached to an air powered winch located on surface at the collar. The pump was connected to two, 20 foot long lengths of 4 inch pipe. A flexible hose connected the 4 inch pipe to the 6 inch pump column. This pump was used (with air boosters) to a depth of 300 feet.

Next, the Flygt BS-2400 pump was installed and it was used down to the 850 foot Level. During this time, the CIR 2GT pump with a surge tank was installed on the 650 foot level and connected to the 6 inch pump column. Once again, air boosters were used to assist pumping.

At the 850 foot level the Flygt BS-2400 pump burned out and was replaced by the Flygt BS-2201. This pump was used to the 950 foot level. At this elevation a holding tank with a Flygt BS-2201 was installed as a temporary pump station and shaft dewatering continued to the shaft bottom using a second Flygt BS-2201 which discharged into the 950 foot level holding tank.

Pumping started on December 13, 1987 and the mine was dewatered to the 1250 level by May 30, 1988. It has been estimated that 3,500,000 gallons of water were pumped. This water was discharged on surface into a sedimentation pond. The overflow was filtered before flowing into Barber Lake.

During the dewatering period the top of the water was kept approximately 10 feet below the last set being replaced.

The major problem associated with pumping was the burning out of pump motors due to the dirty water, the number of starts and stops and low voltage due to the diesel generator.

At the time of writing (November, 1988) the mine was making water at a rate of 25 gpm. The pumping system consists of a CIR 2GT pump with dirty water/clean water sumps on the 1250 level and a similar pump with a surge tank on the 650 level.

Shortly after pumping started, work began on replacing the old timber sets with concrete rings and timber dividers. (This work progressed simultaneously). The procedure for doing this is illustrated in Fig. 5. The sequence that was followed is given below.

1) Guides were removed and the working platform was lowered to the top of the timber set. Walls were scaled and secured (if necessary).
2) Plywood forms were installed outside of the timber sets and posts. A shiplap floor was installed on top of the timber set and the space between the floor and the rock wall was chinked with burlap.
3) The steel reinforcing was installed behind the forms and the forms were oiled with form oil.
4) Next, the concrete was lowered in a concrete bucket and placed behind the forms using a 8 inch diameter “elephant trunk” hose. Concrete was vibrated into place using air powered vibrators.
5) Once the concrete had set, the forms were stripped, the posts removed and the new timber dividers installed. Next, the guide brackets, guides, service lines, manway landings, ladders and screen brattice were installed.

   This process was repeated for each set.

Work progressed three shifts per day with concrete usually being poured on day shift and occasionally on afternoon shift.

Set replacement started on January 6, 1988 at Set 7 and was completed June 3, 1988 at Set 199. (Fig. 6) (The first three sets were in the collar pour) A total of 204 sets were replaced in 141 days for an average of 1.37 sets per calendar days.

Approximately 700 cubic yards of concrete were poured. During this period there were several delays due to pumping problems.

During the refurbishing period the Phase 2 sinking plant was also being constructed. This included installation of a 10-foot diameter double drum sinking hoist, installation of permanent electrical system and the construction of a 105 foot high headframe. This headframe had to be constructed over the portable headframe. In order to be used for sinking, bucket dump doors were installed 35 feet above the collar. During the sinking phase the headframe bin will not be required. Instead, the broken rock will be dumped on the ground and transported by front-end loader to provide fill at the site.

The shaft sinking gear consists of a steel blasting set suspended on 3/4 inch cables from two winches located on the 1250 Level. Two 6 ton buckets and crossheads will travel in the hoisting compartments. During mucking, three buckets will be in use. One at the bottom being filled and two travelling in the shaft.

The shaft will be sunk using the benching method. A total of 30 holes, 1 1/4 inches in diameter will be drilled on each bench. Each bench advances half the shaft approximately 6 feet. CIL Forcite explosives and Magnadet primers will be loaded and all firing will be from surface. Exhaust fumes will be removed by fresh air blown to the shaft bottom via a 24-inch rigid fiberglass. duct. The ventilation fan will be a Woods, 40 H.P., 24 Kg fan.

The broken rock will be mucked with a Cryderman shaft mucking machine. This machine has a .5 cubic yard capacity bucket. The Cryderman is positioned below the manway compartment and is suspended on a 3/4 inch cable attached to a 25 HP electric winch on the 1250 Level.

Shaft sinking started on September 5, 1988 and at the time of writing (November, 1988) the bottom was advancing at a rate of 7.5 feet per day.

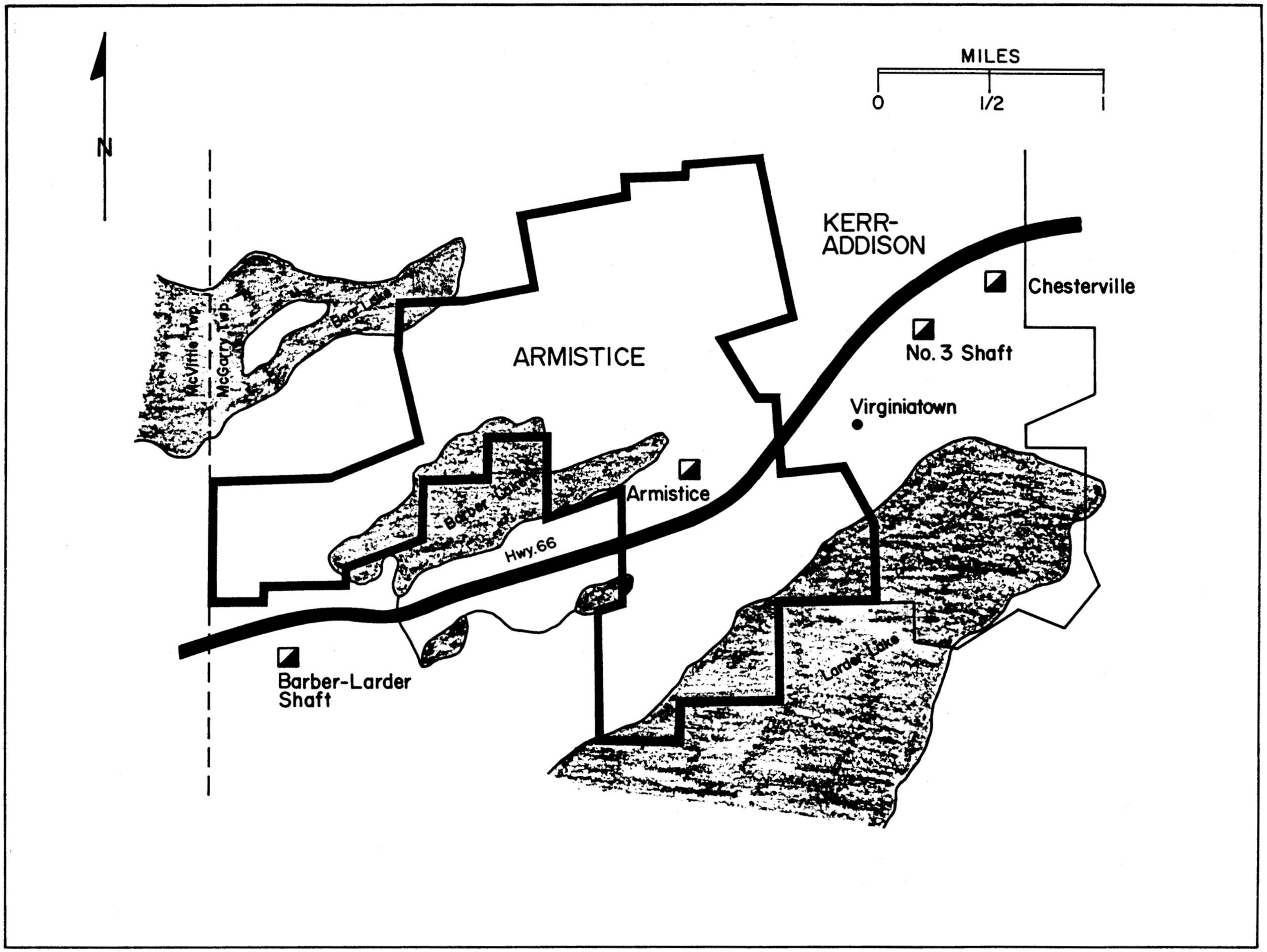
The authors would like to thank Armistice Resources and Dynatec Mining Limited for granting permission to write this paper. They would also like to thank the staff of Dynatec, Jack de la Vergne, Consultant and Murdy Armstrong, Armistice Resources for their assistance during the planning stages of the project and the preparation of this report.

REFERENCES

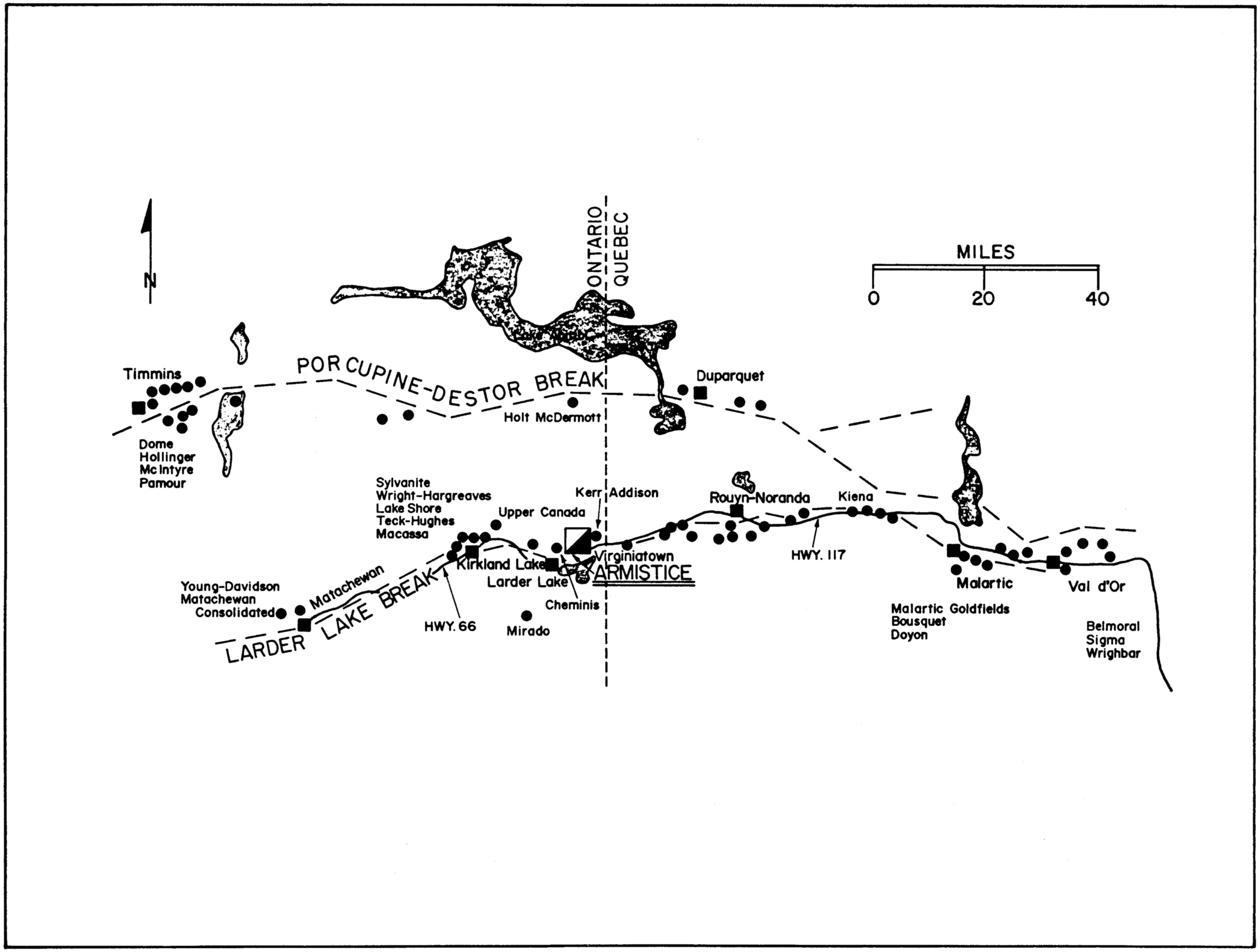
1) Jack de la Vergne; A Guide to the Selection of Hoists for Shaft Sinking; International Conference on Hoisting—Men, Materials, Minerals; Canadian Institute of Mining; June 1988.

BIBLIOGRAPHY

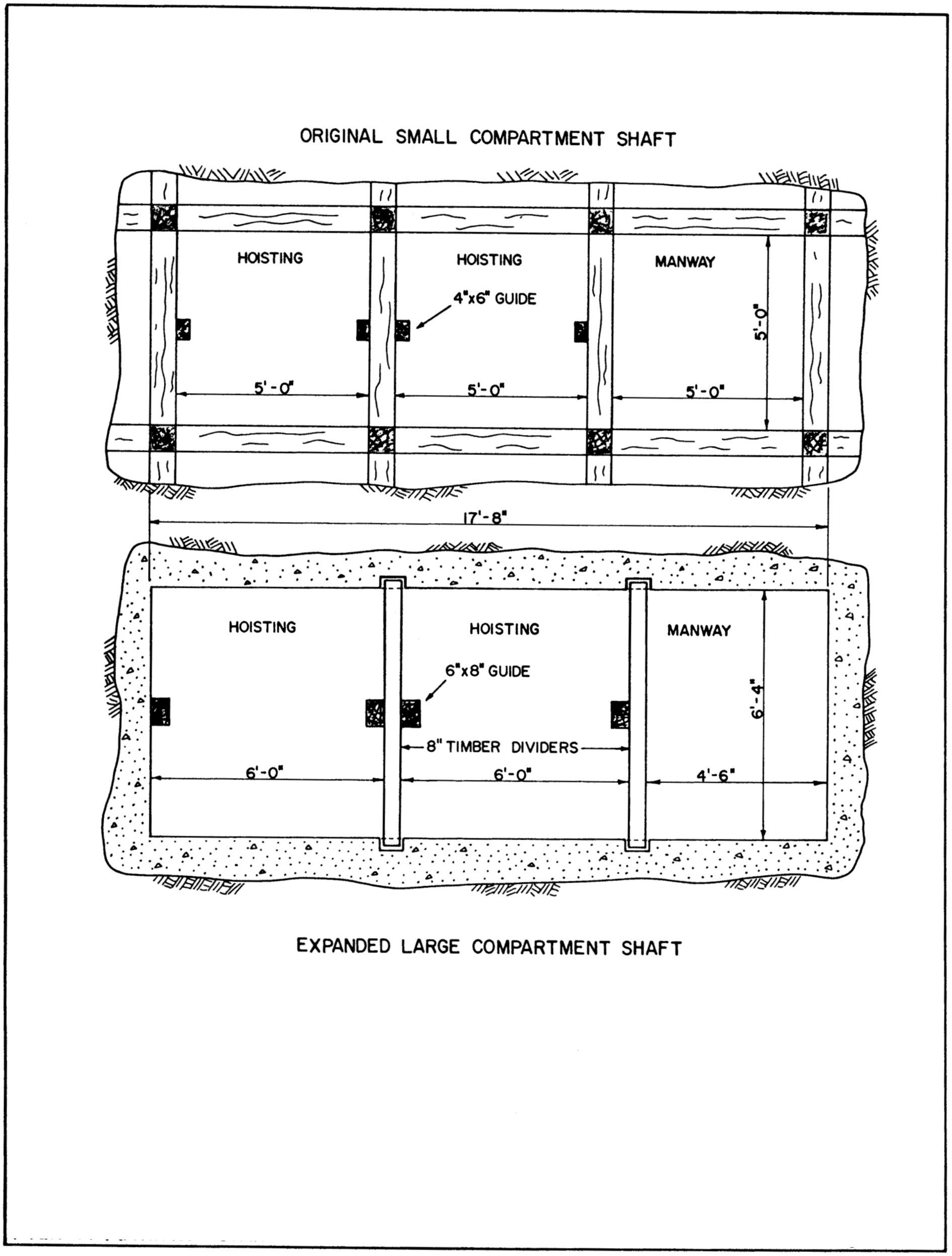
1) Report on the Exploration and Development, Sheldon-Larder Claims, Armistice Resources Limited, McKay Mining Engineering Inc., May 29, 1987.
2) Armistice Resources, 1987, 1988 Annual Reports.



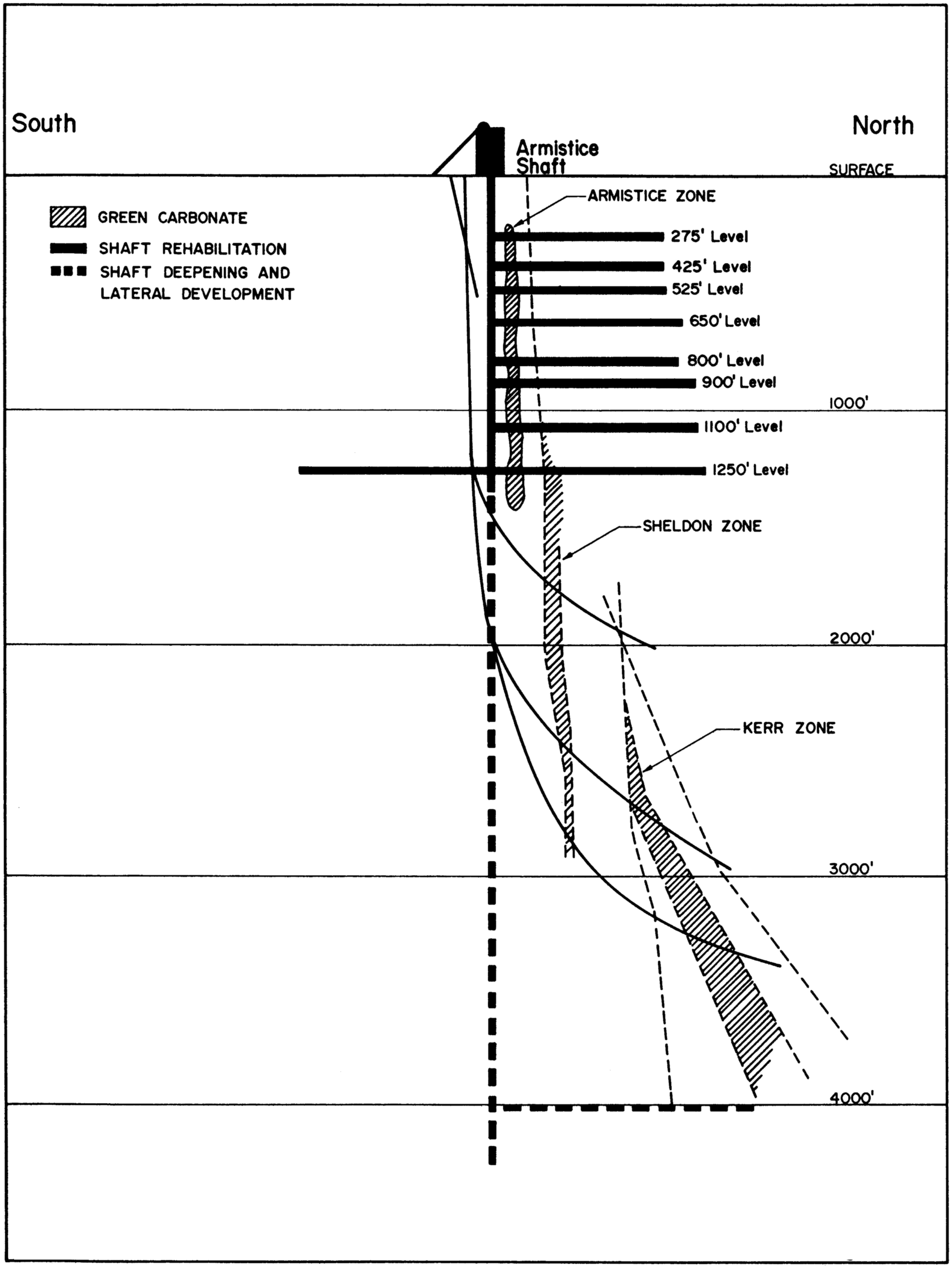
Location of Armistice Claims Figure 1.



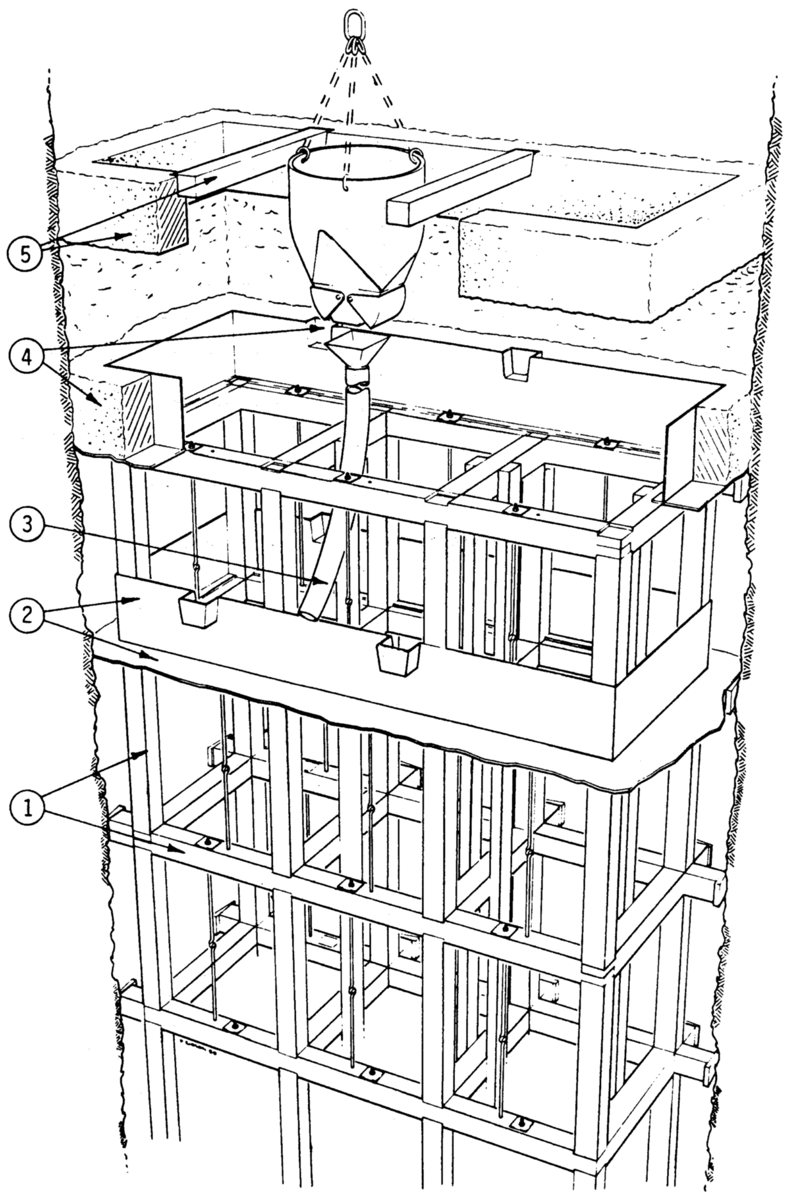
Location of Armistice Claims Figure 2.



Plan of Armistice Shaft Figure 3.

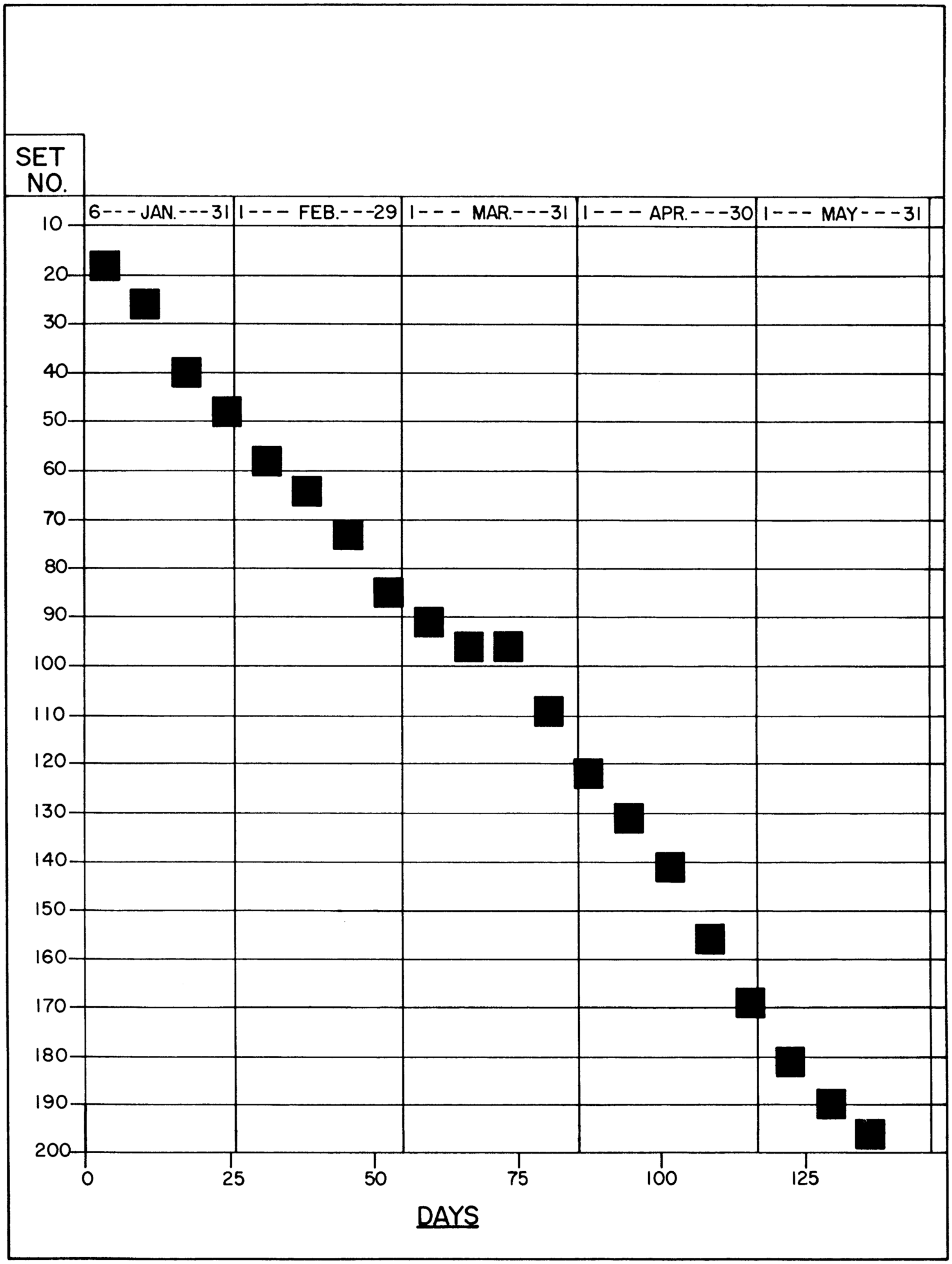


SECTION 1300W (looking west) Figure 4.



1. Original shaft timber sets.
2. Install forms outside timber sets with plywood floor between set and rock wall for a 2' high ring.
3. Pour concrete behind forms.
4. Concrete ring showing forms and pocket for wooden dividers.
5. Concrete ring after forms are stripped and dividers in place.

Timber Set Replacement Sequence Figure 5.



Set Replacement Schedule Figure 6.

Stability of liners in shaft design

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SYNOPSIS

In some recent shaft constructions the shaft-linings consist of two different materials, an inner metallic cylinder and an outer concrete or reinforced concrete wall. In most cases the inner cylinder (in the following called: liner) is relatively thin-walled with respect to the concrete bedding. In some cases this "composite construction" is enclosed by a water-tight welded steel membrane.

Some examples for such constructions in West-Germany are freezing shafts for carbon and salt mining and drilling shafts.

The topic of this paper deals with the stability of the metallic liner inbedded in a nearly rigid circular wall subjected to outer mountain—or water-pressure perhaps in connection with heating.

The effect of the hydrostatical loads must be distinguished as follows:

1. If the concrete cylinder is assumed to be permeable, the water—or asphalt-pressure will act upon the back side of the liner. This case is called "water-pressure-problem".
2. If the concrete cylinder is water-tight (perhaps by an outer welded membrane) the pressure will act upon the outer surface of the whole composite construction. Because thin liners still tend to stability failure, this case is called "composite-pressure-problem".

For the statical analysis of liners two different ways are described: The analytical solution of the differential equations of the problem and the simulation by a special framework and finite element analysis.

Finally, the influence of different imperfection geometries and non-rigid beddings are discussed and compared with the results of Link, Glock, Hain, et.al. The stiffening effect of shear and tension bolts and the nonlinear physical behaviour are shown in pictures and diagrams.

INTRODUCTION

Two walled shaft sinkings (sliding shafts) are used to separate mining influences from the inner water-tight cylinder. The principle can be seen in Fig. 1: The gap of some 150 mm between the outer and the inner cylinder is filled with asphalt to admit relative movements without risks to the inner cylinder, which will be the subject matter of this paper.

The increasing depth of the water-bearing formations in German carbon mining has produced new constructions for the inner wall. Down to some 400 m, a homogeneous inner concrete wall would be sufficient and economic. An outer steel cover with normally 8 mm wall-thickness prevents the liquid from penetrating the concrete wall. For deeper shafts, *composite constructions* have proved to be advantageous. The cross section of the inner cylinder is reinforced by a liner construction made of welded steel or cast iron segments with bolted flanges. If the liner has an inside position, the outer membrane and the liner can be used as a form for the concreting, see Fig.2. Recent shaft sinkings with composite sections have been Voerde (600 m) and Sophia Jacoba 8 (560 m)[1].

Other examples for liner constructions are shafts for the salt mining[9], which have become leaky and must be repaired with an inner metallic cylinder. In this case, the space between the shaft wall and the liner is filled with concrete.

Stability problems of liners also arise in drilling shafts, which are wellknown in the USA[2]. The inner steel cylinder is stiffened by welded steel bandages and has a concrete bedding as described already.

For the stability analysis, the threedimensional problem of the shaft construction will be idealized as a twodimensional system and the outer constant area pressure [F/L^2] becomes a radial line pressure [F/L].

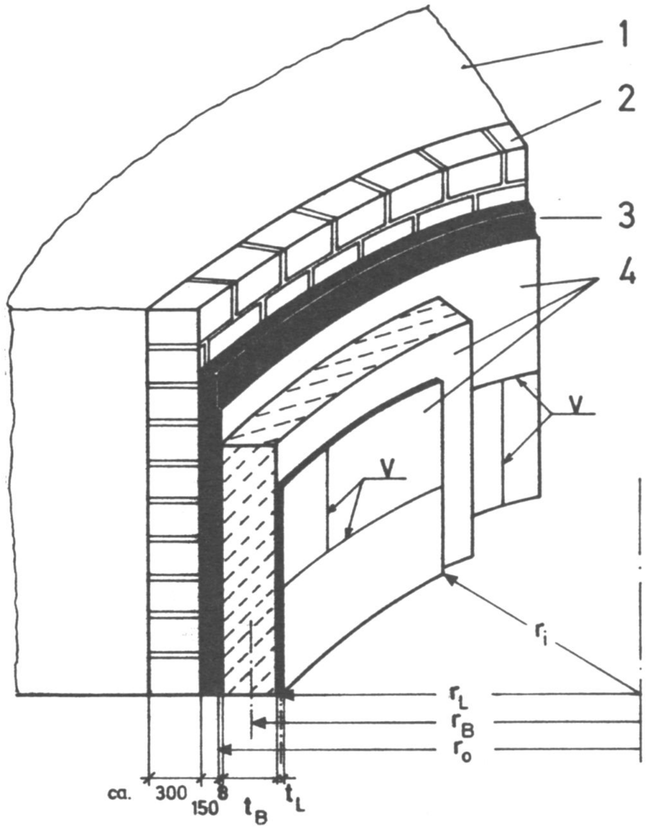

Fig. 1 Sliding shaft construction:

1 water-bearing soil strata

2 outer wall (concrete blocks)

3 gap filled with asphalt

4 inner wall: steel membrane, concrete cylinder and liner

<u>List of used Terms</u>

Index L	terms related to the liner
Index B	terms related to the concrete bedding

geometrical terms:

r_o	outer radius of the steel membrane
r_L	radius of the liner axis
r_B	radius of the concrete bedding axis
t_L	wall-thickness of the liner
$k^*=r_L/t_L$	relation of radius and wallthickness of the liner
w	radial deflection

loading terms:

Po	liquid pressure acting on the membrane
P_L	loading of the liner
P_B	loading of the bedding
P…or	critical loads of perfect liners
P…D	critical loads of imperfect liners
index W	water-pressure
T	increase of temperature (heating)
VW	prestressing+water-pressure

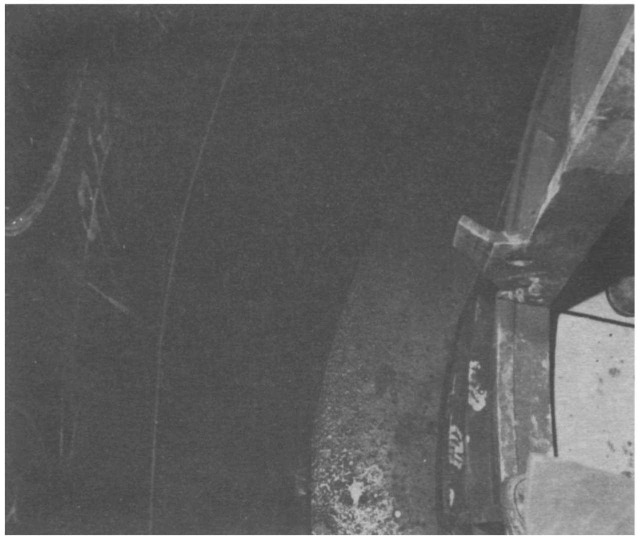

Fig. 2 Steel membrane and liner used as a form for the concreting

V	composite-pressure
$\alpha = pr3/EI$	dimensionless load parameter (indices as p)
W_v	amplitude of the imperfection
$\varnothing_1$	region of snap-through, border of the imperfection
F1	disturbance force at bents of liner segments

COMPARISON OF THE STABILITY PROBLEMS

Load Behaviour

For the following analysis three separate load cases are investigated (see Fig. 3) :

a) The bedding is permeable to hydrostatic forces and the full pressure Po acts upon the outside of the liner. This case will be called the “water-pressure-problem”.
b) The liner is heated, while the temperature of the bedding is assumed to stay constant. As the liner cannot enlarge within the rigid bedding, contact pressure p_L will act upon its outside. This case will be called the “heating-problem”.
c) The bedding is impermeable or surrounded by an outer water-tight membrane. The hydrostatic pressure p_o acts upon the outer surface of the composite construction, but both rings take part in the pressure p_o: The bedding gets P_B and induces the contact pressure p_L to the outside of the liner. This case will be called the “composite-pressure-problem”.

The contact pressure p_L can be calculated in case b) from the identity of pressure- and temperature-strains:

$$\epsilon_P = p_L r_L / E_L A_L = \epsilon_T$$

gives

$$p_L = \epsilon_T \cdot E_L A_L / r_L . \qquad (1)$$

In case c) the radial shortening w of the two rings must be equal. With

$$w = \epsilon r = pr^2 / EA$$

using the indices B and L for bedding and liner we get:

$$p_B r_B{}^2 / E_B A_B = p_L r_L{}^2 / E_L A_L$$

and from the condition of equilibrium with the outer pressure p_o:

$$p_B = p_o r_o / (æ_P r_L + r_B)$$

$$p_L = æ_P \; p_B \qquad (2\text{ a, b})$$

$$\text{with} \quad æ_P = (r_B/r_L)^2 \cdot E_L A_L / E_B A_B$$

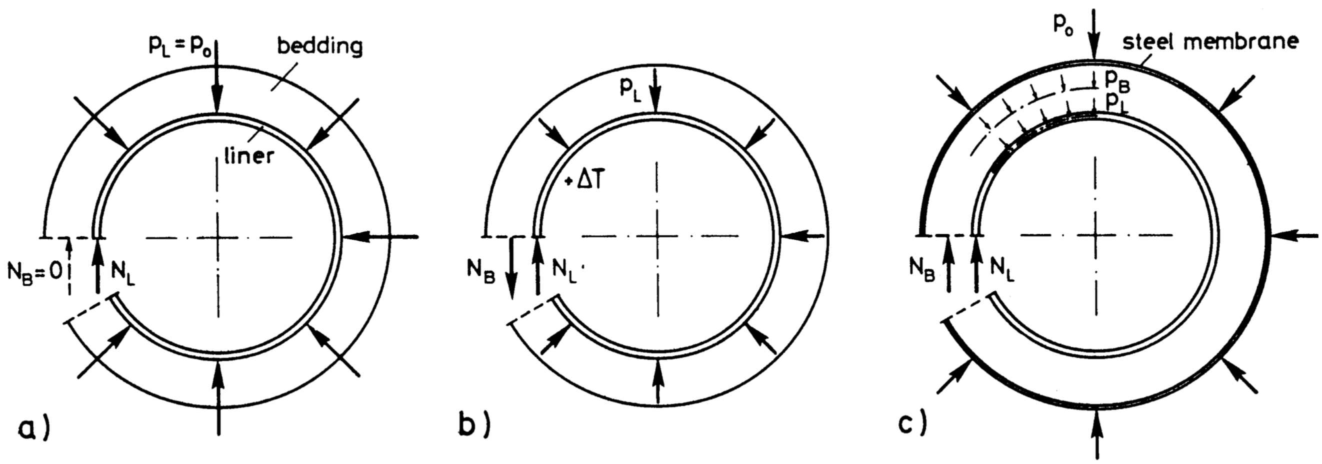

Fig. 3 Subcritical loading ($p«p_{or}$):

a) hydrostatic pressure p_o acting upon the liner

b) heating of the liner

c) composite-pressure P_L caused by outside hydrostatic pressure po

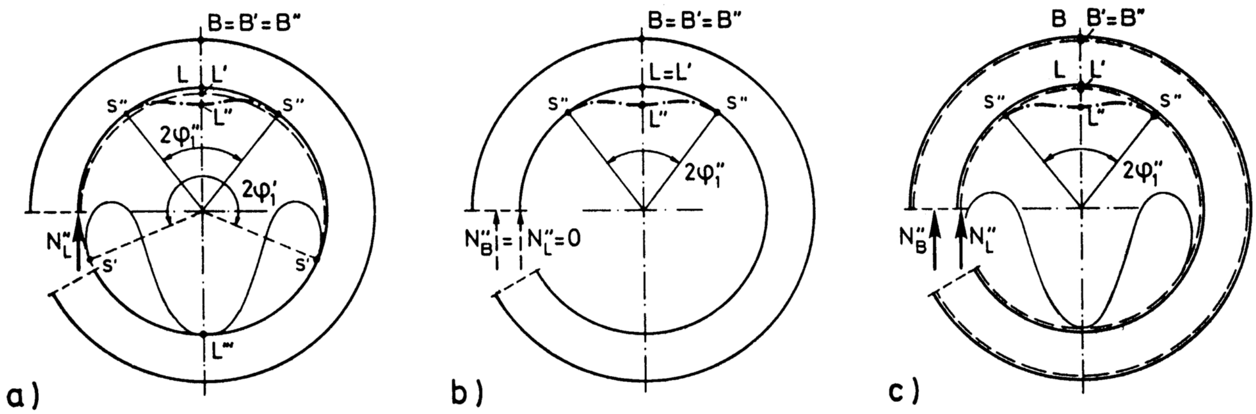

Fig. 4Deformations of the liner:

…' state before buckling;

…" state after buckling;

a)−c) see Fig 3

In the following discussion about the deflection behaviour of the liner until buckling, it is of utmost importance to look for the normal forces that arise in the composite construction. In the initial state—defined by loads which are much smaller than the buckling load —the normal forces N_L can be calculated from the load p_L by the formula

$$N_L = p_L \; r_L \tag{3}$$

for cylindrical tubes under hydrostatical radial pressure.

In all three cases we get negative normal forces for the liner, but different results for the bedding:

case a)	N_B=0,	
case b)	N_B>0	(tension force),
case c)	NB<0	(pressure force).

Heating-problems like case b) cause residual stresses with a zero integral over the complete wall-thickness. Heating is completely analogues to the load case "prestressed ring", where a tension force N_B>0 is brought into the outer cylinder and the liner is compressed. This analogy will be helpful in the next chapter.

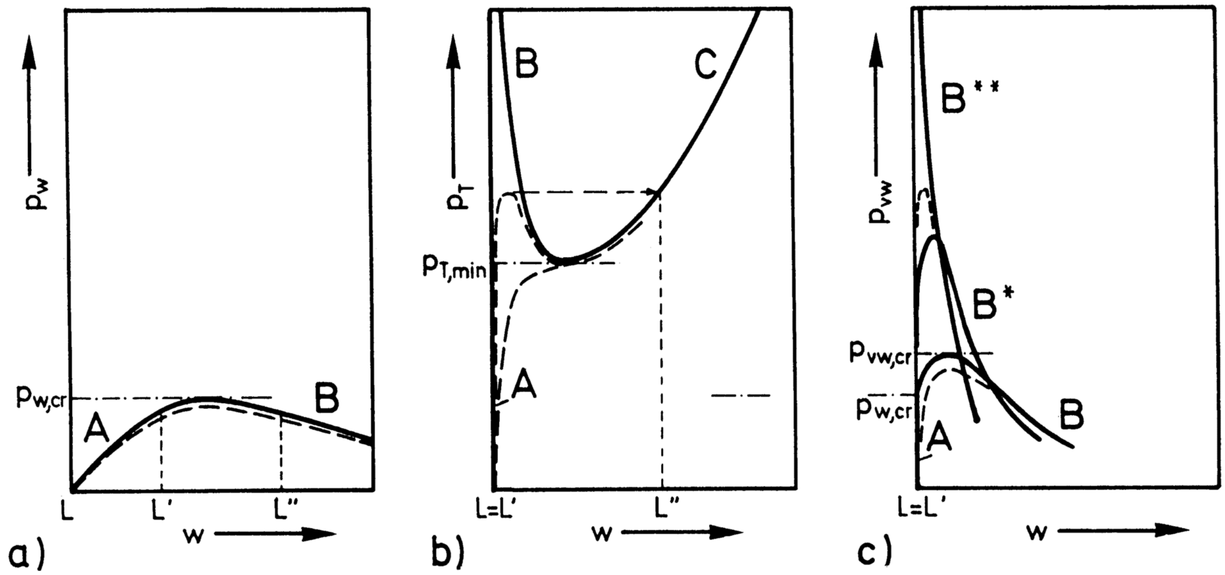

Fig. 5 Load-deflection-curves of the liner: a), b) see Fig. 3

c) prestressing and water-pressure, a first approach of the composite-pressure-problem

Deflection behaviour and buckling

Until now, the initial state of the loading has been described. When the hydrostatic loads on the outside are increased, the system reacts by greater section forces and deflections. We distinguish between the state before (…') and the state after the loss of stability (…"). The normal forces and the deflection functions of the two states are drawn into the system figures of the shaft cross sections, see Fig. 4.

The liner under *water-pressure* (case a) reacts by a shortening of the circumference—the liner is separated from the bedding in a great region $2\varnothing_1'$. The region where the gap arises is arbitrary and assumed in the crown of the liner at the point L. At higher pressure P_o the separation point S goes upward (decreasing angle $2\varnothing_1''$ a phenomenon called "rolling up" in the literature[3],[4]. The crown region becomes flat and the deflection of point L increases more rapidly until a buckle of the arch length $r_L \cdot 2\varnothing_1''$ springs into the circle and the system collapses (L"). This process has an analytical description in the load-deflectioncurve (Fig.5a): The deflections w (=L− L') grow more rapidly than the load p_w and reach (theoretically) infinity at the buckling state, marked by the maximum $p_{w,or}$ of the load-deflection-curve.

In the prebuckling state of a *heated* or *prestressed* ring (case b) no deflection can be observed at all. The chosen crown point L' remains in the original position L, when the liner is thin-walled with respect to the bedding.

Buckling happens very suddenly and noisy: A buckle within the region springs into the circle, but stops at L". A second stable equilibrium state has been reached, where the axis of the liner is free of strains (see N_L''=O in Fig. 4 b). A further increase of temperature or prestressing would be possible, if reasonable. This buckling phenomenon is called "snap-through" in the literature.

In many publications[5,6…] the question has been discussed, if critical temperature loads could be derived and used for practical design.

The problem of the *elastically* bedded and prestressed ring with the critical load[4],[7]

$$\alpha_{cr} = p_{cr}\, r^3/EI = 2\cdot(1+cr^2/EA)\cdot\sqrt{cr^4/EI} \qquad (4)$$

gives a good insight in the treated load case. If the stiffness c of the radial springs is enlarged to simulate a *rigid* bedding, the critical load becomes infinite. This means that no analytical values for the snap-through-effects of perfectly round liners exist, although they have been observed in experiments.

An explanation can be given looking at the complete load-deflection-curves of the heating or prestressing problem, see Fig. 5. The prebuckling state without deflections of the crown (L=L') is drawn as a thick vertical line through the origin (line A). The buckled states have been calculated by nonlinear analyses[4,8] as some parabola with a descending branch B close to the vertical line A (but not touching it), a minimum and an ascending branch C belonging to great deflections (L"). It is obvious, that for

1) $p_T < p_{T,min}$ no buckling is possible

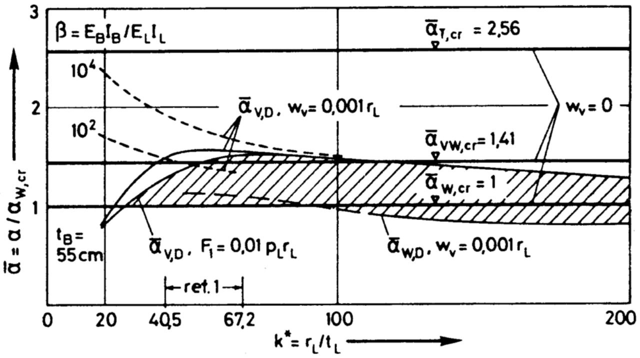

Fig. 6 Comparison of the critical loads for perfect and imperfect liners

2) $p_T > p_{T,min}$ buckling might happen, if any disturbances are inherent or imposed to the system.

In the last case the system goes the dotted horizontal line from stable equilibrium A, passing the instable branch B and reaching a second stable state of equilibrium in branch C. It is obvious, too, that for loads much greater than $p_{T,min}$ only very small disturbances are sufficient for system failure. This corresponds with the sudden and noisy snap-through observed in experiments.

After having referred to the two already known problems "water-pressure" and "heating or prestressing" we now come to the yet unknown *"composite-pressure-problem",* case c. From Fig.4 c) it can be seen that there is no gap between the bedding and the liner in the prebuckling state: The diameters of all parts of the shaft cross sections shorten (B becomes B' and I, goes L'). Under this aspect the new problem c is similar to problem b. On the other hand, in the postbuckling state of problem c the section forces N_B" and N_L" are still necessary for equilibrium with the outer pressure p_o. Now it seems like problem c belongs to case a. It can be summarized, that the "composite-pressure-problem" consists in the "water-pressure-" and the "heating-problem".

A look at the load-deflection-curves of case a and case b shows totally different behaviours as discussed above. As a first approach to connect a and b and to get insight into the compositepressure-problem the following load-case is defined:

"prestressing together with water-pressure"

Fig. 5c) shows the resulting load-deflection-curves. The thick curve was calculated for the load combination $p_v = p_{w,or}$ and additional water-pressure. The magnitude of $p_{w,or}$ was taken from Fig. 5 a). The numerical solution was developed from the equations in Link's recent publication[9].

This first approach to the composite-pressure-problem is sufficient with respect to the following qualities: The curve c) shows a vertical state A like curve b) and a continuous decrease in the postbuckling state B like curve a). Unfortunately, the load definition was arbitrary: greater prestressings of the liner cause greater critical loads $P_{vw,or}$ up to infinity (see curves B* and B**)—just similar to the heating-problem b!

Critical loads of perfect liners

As the critical loads for the water-pressure—and the heating-problems are known and a first approach for the composite-pressure-problem could be proposed, it is interesting to compare the results in a greater region of radius-wallthickness-relations $k^* = r_L./t_L$.

In ref.[4,6,9,10] the following critical load parameters for perfect liners have been derived:

$$\begin{aligned} &\text{case a)} \quad \alpha_{w,cr} = 2{,}62\, k^{*0{,}8} \\ &\text{case b)} \quad \alpha_{T,min} \geq 6{,}72\, k^{*0{,}8} \\ &\text{case c)} \quad \alpha_{vw,cr} = 1{,}41 \cdot \alpha_{w,cr} \\ &\qquad \text{with } \alpha = pr^3/EI \end{aligned} \qquad (5\text{ a-c})$$

If they are devided by equation (5 a) we get the factors 2, 56 for case b) and 1, 41 for case c), which are drawn as thick lines parallel to the horizontal axis in Fig. 6. Although the results seem plausible with respect to the mechanical foundations discussed above, they are not sufficient from safety aspect. An *imperfection concept* must be added to establish safe design rules for composite shaft constructions.

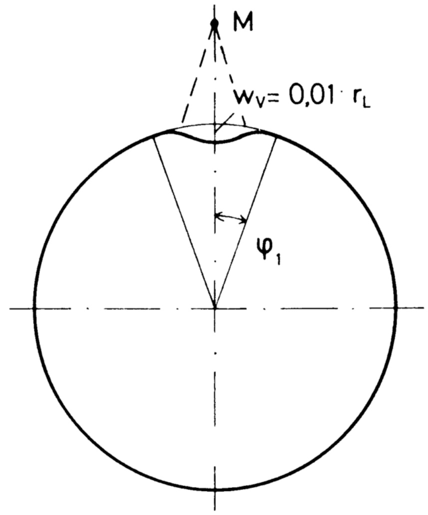

Fig. 7 Local geometrical imperfections of the liner (enlarged drawing of the function w_v)

IMPERFECTIONS OF LINERS

Imperfection sensitivity

In the German steelwork-rules for the design of pressure members or frameworks DIN 18800 p.2[11], two kinds of imperfections are mentioned:

- structural imperfections (i.e. residual stress, variations of the modulus of elasticity),
- geometrical imperfections (deformations of the structure, which result from welding, erection or other influences).

The load-deflection-curves are influenced for example by geometrical imperfections, because they cause bending deflections in the initial state A. The values w become relatively greater and the buckling loads decrease, see the dotted lines in Fig. 5 a-c).

From many stability researches in the past (starting with the postbuckling theory of Koiter) it is well-known today, that the function of the loaddeflection-curve in the pre—and postbuckling region (A, B and perhaps C) has a great influence on the reduction of the critical load when imperfections are present. This phenomenon is called the "imperfection sensitivity" of the system. If the branches A and B are steep and close together, the system will be very sensitive with respect to imperfections (Fig.5 c), line B**). Whereas if the branches A and B are flat like in case a), the reduction of the critical loads will be moderate (in this case some caution is necessary regarding the scale of the w-axis).

The heating-problem (case b) shows high and low sensitivity as well: Small imperfection will produce the upper dotted curve (representing a sharp snap-through-effect) and great imperfection the lower smooth curve. However the load $P_{T,min}$ is conservative and can be seen as an absolute minimum for all possible imperfections.

Practical imperfections

As can be seen it is necessary to take imperfect liners into consideration. The next problems to discuss are the geometry and the amplitude of imperfection functions. It is suggested to adapt them to the *buckling modes* of the problem. This method is in accordance with today's stability rules[11] to produce results that are on the safe side. On the other hand, the imper-fections should be realistic with respect to practical erection methods.

Following this concept, two different kinds of imperfections are proposed[10]:

1) For liners made of bent and welded steel sheets: *local deformations* within the region (=total angle of separation in the moment of buckling) and the function

$$w_v = w_v(\phi=0°)\cdot\cos^2(\pi\phi/2\phi_1) \qquad (6)$$

drawn in Fig. 7.

2) For liners made of steel or cast iron bolted segments: deviations of the theoretical radius, which cause bents in the connection points, see Fig. 8,9. Introducing f as half of the diameter deviation (d_{max}–d_{min}), a radial *disturbance force*

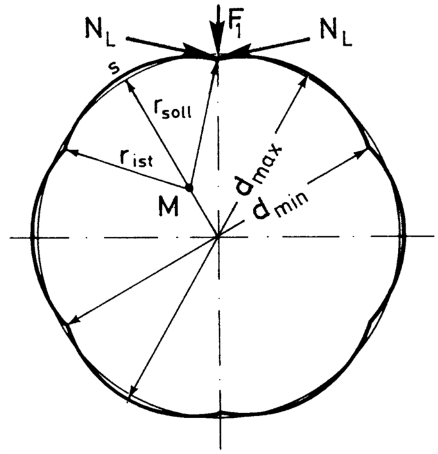

Fig. 8 Disturbance force caused by radius deviations

$$F_1 = 8\, N_L f/s \quad \text{with } N_L = \text{liner normal force}, \quad s = \text{arch length of the segments} \tag{7}$$

can be calculated.

For case 2) it is on the safe side to place the force F_1 only at the crown and to admit tangential movements of the liner beyond the crown when the liner is not anchored in the circumferential direction (liners with shear anchorage see later).

Both kinds of imperfections support a one wave buckling mode. Multi-mode buckling has neither been observed in theoretical nor in experimental researches[4,6] when the bedding is rigid. It must be mentioned, however, that for elastically bedded liners multi-mode buckling might be relevant!

In the following, buckling loads are derived for liners with local deformations as well as disturbance forces for comparison reasons and safety aspects.

SOLUTION METHODS

Analytical solutions

In ref.[12] a system of six nonlinear differential equations for the elastically bedded ring has been derived. They are valid for large displacements and inclinations but small strains (physical linearity). Imperfections given from eq. (6) have been taken into account. The present problem of a rigid boundary is solved by increasing the stiffness of the circumferential springs. Numerous calculations[4] have confirmed the results of the water-pressure- and heating-problem, eq. (5a) and (5b).

The analytical method is limited, when complicated load cases or boundary conditions have to be treated.

Simulation by framework analysis

For the composite-pressure-problem the double-ring-system in Fig.10 is chosen: The inner ring represents the metallic liner (radius r_L), the outer ring forms the concrete bedding (radius r_B) and the radial members guarantee an equal distance between the rings in the bedded region. Very small strain stiffness of the radial members in the separation region $2\phi_1$ admits inward deflections. Because of symmetry, half the system is sufficient.

The coordinates of the first three or four liner nodes content the geometrical imperfections from eq. (6) within the angle ϕ_1 (see Fig. 10, a large scale was chosen to emphazise the imperfection function). If the disturbance force from eq. (7) is relevant, F_1 is placed on the crown point of the liner.

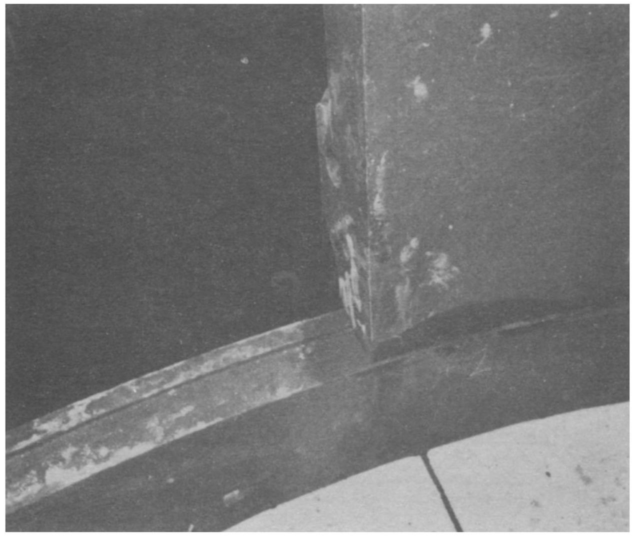

Fig. 9 Practical imperfections of liner segments

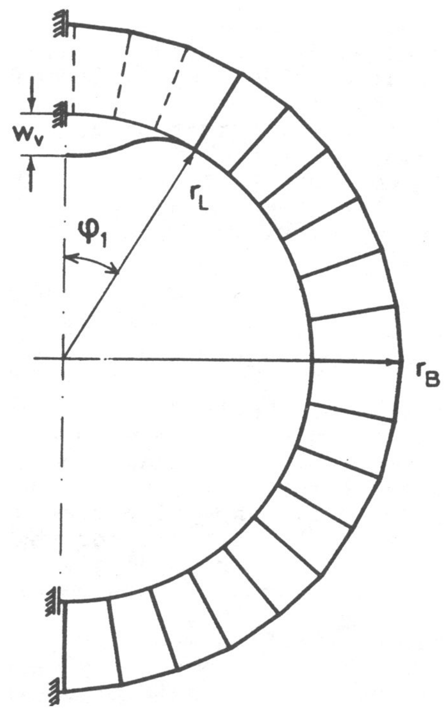

Fig. 10 Framework simulation for the composite construction

The total loading p_o is divided into P_B and p_L by eq. (2a, b), substituted by single forces and placed on the nodes of the outer and inner ring. Increasing the forces step by step, the crown of the imperfect liner goes inward until the deflections become infinite and the construction collapses.

As the buckling loads of perfect liners under water-pressure are already known from eq. (5a), an adequate framework system can be used for calibrating. It can be summarized, that a simulation of the liner

a) by *straight* members[13] produces about 10% higher buckling loads than eq. (5a), whereas
b) by *curved* members[14] exact critical loads result.

For large range parameter calculations the stiffer model a) has been used to reduce expenses.

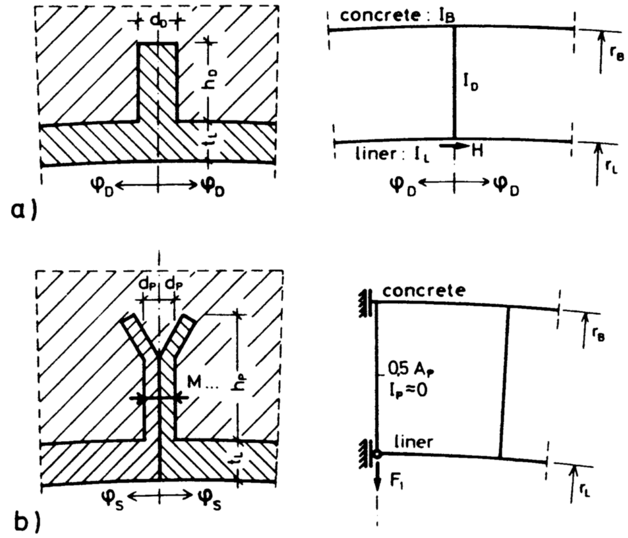

Fig. 11 Simulation of liners with a) shear anchorage b) tension anchorage

RESULTS

Not anchored liners

The maxima of the load-deflection-curves belonging to the composite-pressure-problem are drawn in Fig. 6 for parameters $20 \leq r_L/t_L \leq 200$ and $w_v/r_L=0{,}001$, respectively $F_1/N_L= 0{,}01$. The buckling loads for the water-pressure-problem are also given for comparison reasons. It can be seen, that

1) the imperfection sensitivity increases with decreasing wall-thickness—a plausible result, but not obtained from eq. (5c), drawn as a horizontal line in Fig. 6,
2) for small geometrical imperfections $W_v/r_L=0{,}001$ the buckling loads of the two problems are different—for composite-pressure it would be not economic to use the water-pressure loads $p_{w,D}<P_{V,D}$,
3) the two chosen imperfections cause nearly the same buckling loads for $k^*>50$.

The most important result however is the fact, that the critical load parameters $\alpha_{w,or}$ and $\alpha_{vw,or}$ are not valid for *real liners.* Realistic imperfections must always be considered in a safe analysis.

If the amplitude of the geometrical imperfections rises up to $w_v/r_L=0{,}01$ (factor 10), then the differences between $\alpha_{w,D}$ and $\alpha_{V,D}$ become zero and thus no economic advantage remains[10]. The reason for this phenomenon can be seen in the unfavourable shape of the initial buckle: It has a negative curvature as shown in Fig. 7.

Non-rigid beddings

If liners are relatively thick-walled with respect to the concrete cylinder, the bedding cannot be assumed to be rigid. The parameter $ß=E_BI_B/E_LI_L$ is useful for the description of the problem. Fig. 6 shows two lines for $ß= 10^2$ and $ß=10^4$, a third line for a constant wall-thickness t_B=55 cm with varying t_L, including the parameters of a completed shaft[1]. It is evident, that weak beddings cause decreasing buckling loads $p_{v,D}$. Further results see ref.[10]

Liners with anchorage

Liner segment can be produced with ribs and tension anchors, which react on the concrete cylinder and improve the stability behaviour of the whole system. Such anchors will be introduced in the theoretical model Fig. 10 in a simple way: Shear elements are simulated by radial members with defined bending stiffness, which reduce the tangential deflections (Fig.11a). Radial anchors like fig. 11b) can be described by springs (axial stiffness of the relevant radial members). The spring stiffness is derived from experiments or theoretical considerations.

The effect of such anchors is shown in Fig. 12. To be on the safe side, a *hinge* has been introduced in the crown of the liner (from the above discussion it is known, that one hinge is sufficient). The hinge reduces the buckling load dramatically, but the shear anchorage (I_D>0) will increase it again. Adding a tension anchorage, the buckling load becomes still greater[10].

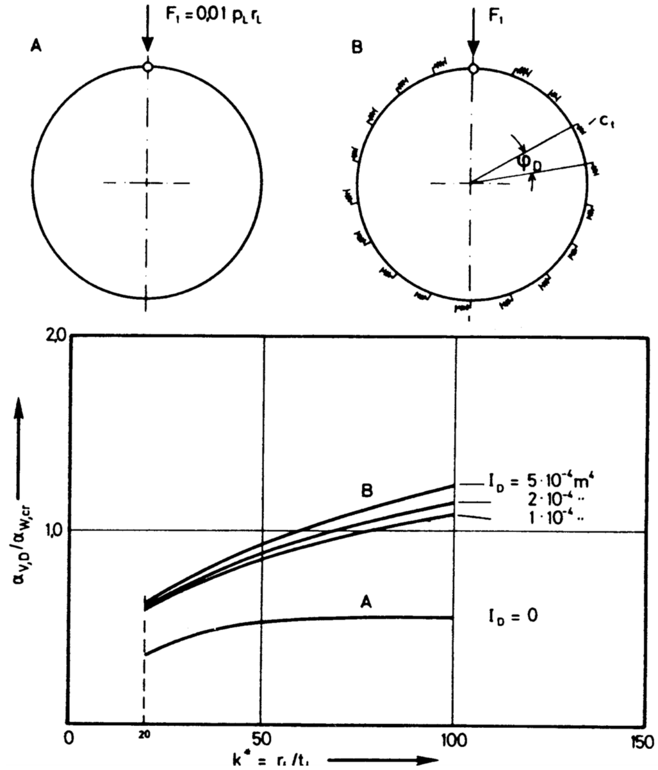

Fig. 12 Buckling loads of liners with a hinge and shear anchorage

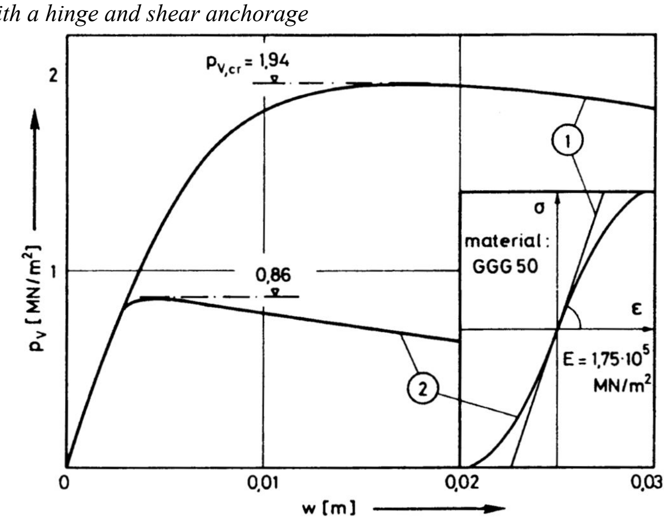

Fig. 13 Load-deflection-curves of a liner with linear and nonlinear material behaviour

Material nonlinearity

The stress-strain-relations of the liner material (cast iron or steel) are obtained from tension/pressure and bending tests. When the liner is relatively thick-walled, the inner and outer fibres of the cross section get nonlinear strains. In the finite element analysis[14], a special layer element admits nonlinear stress-strainrelations. The result is a decrease of the buckling load when compared with the ideal elastic material behaviour, see Fig. 13.

SUMMARY

In shaft constructions, sometimes an inner liner is necessary for design purposes. The liner is loaded either directly by hydrostatic pressure or by the contact pressure of the bedding. The buckling formulas for the critical pressure of *ideal round* liners are represented. For *real* liners an imperfection concept is necessary to develop safe design rules. Finally, the validity of the simulation model is discussed with respect to flexible bedding, anchorage and material nonlinearity.

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References

1. Klein, J.; Rieß, H.-G.; Ritter, H.: Abteufen und Ausbauen des Gefrierschachtes Sophia-Jacoba 8, S. 1395– 1406. Essen: Glückauf 1987
2. Link, H.: Berechnung ringversteifter Bohrschachtverrohrungen aus Stahl in den USA. Glückauf-Forschungshefte 41 (1980) 89–98, Nachtrag (1980) 271–272
3. Glock, D.: Überkritisches Verhalten eines starr ummantelten Kreisrohres bei Wasserdruck von außen und Temperaturdehnung. Stahlbau 46 (1977) 212–217
4. Falter, B.: Grenzlasten von einseitig elastisch gebetteten kreiszylindrischen Konstruktionen. Bauingenieur 55 (1980) 382–390
5. Mettler, E.: Eine Bemerkung zur Frage des Beulens ummantelter Schalen, Bauingenieur 38 (1963) 309–311
6. Hain, H.; Falter, B.: Zum Stabilitätsproblem des starr oder elastisch gebetteten Kreisringes infolge gleichmäßiger Temperaturerhöhung. Pflüger-Festschrift, TU Hannover 1977
7. Sliter, G.E. ; Boresi, A.P.: Buckling of Uniformly Compressed Ring with Radial Elastic Support. Developm. in Mech.3 (1967) 443–450
8. Britvec, S.J.: Sur le flambage thermique des anneaux et des coque cylindriques précontraints. Journal de Méchanique 5 (1966) 409–418
9. Link, H.: Der kritische Außenwasserdruck des starr gebetteten Kreisrohres bei einer Klaffung zwischen Rohr und Bettung. Kali und Steinsalz 9 (1987) 345–348
10. Falter, B.; Klein, J.: Grenzlasten von Linern im Schachtbau. Bauing.63 (1988), to be published
11. DIN 18800, Teil 2: Stahlbauten Stabilitätsfälle. Knicken von Stäben und Stabwerken. Entwurf März 1988. Berlin: Beuth 1988
12. Falter, B.: Berechnung freier und einseitig elastisch gebetteter Kreisbögen und -ringe unter Außendruck mit großen Verschiebungen und Verdrehungen. Diss. TU Hannover 1975
13. Falter, B.: Statikprogramme für Taschen- und Tischrechner, Teil 2. Düsseld.: Werner 1984
14. Hibbitt, Karlsson, Sorensen: ABAQUS. User’s Manual Version 4.5 (a) July 1985

Development of shaft steelwork as applied to deep circular shafts: design and installation aspects

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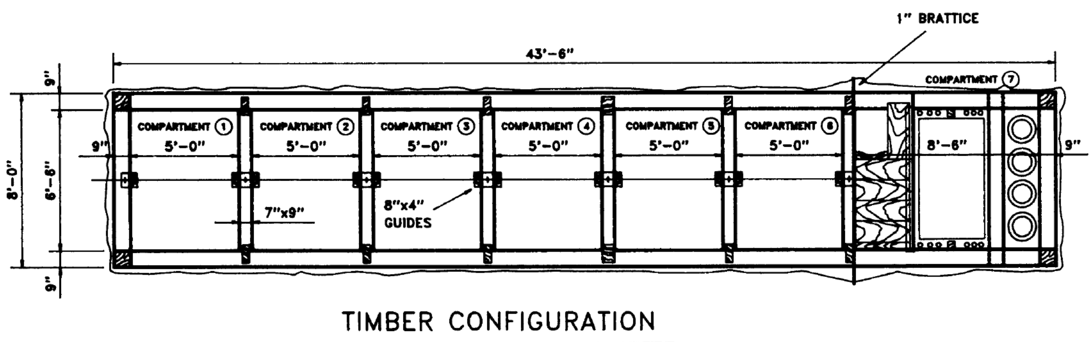


TIMBER CONFIGURATION (Figure 1)

SYNOPSIS

This paper deals with the development of shaft steelwork, buntons and guides as applied to deep circular mine shafts and is written in three sections covering development, design and installation aspects.

It touches briefly on the advancement from the original timber sets and guides through to the present state of the art technology of using hot rolled hollow section buntons and deep section top hat guides. It covers criteria to be applied to new shafts and discusses the background to the design considerations. Connections, fasteners corrosion protection and alternative sidewall fixings are considered.

The shaft installation procedure considers tolerances, jigging, surveying, equipping conveyances, shaft commissioning and alternative equipping techniques.

1.
THE DEVELOPMENT OF SHAFT STEELWORK

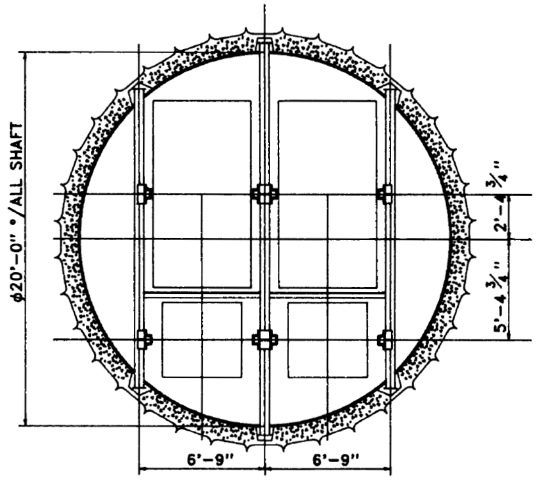
a) Preamble

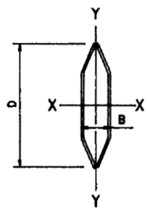
The South African Gold Mining industry in its demand for increased productivity and safety from greater shaft diameters and depths, has ensured that the design of shaft steelwork systems has had to keep pace and in fact forge ahead and become the leading edge technology in its field of the industry. This technology has seen the advancement from the original timber sets to the modern bunton sets of today.

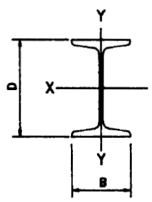
b) Timber Equipped Rectangular Shafts

The early South African shafts were predominantly rectangular and timber equipped. The seven compartment shaft comprising two rock, two man and material, two service and one bratticed ventilation compartment (See Fig.1) gradually gave way to steel equipped rectangular shafts with steel sets at 4 metre intervals as opposed to the timber sets at two metre centres. The ever increasing demand for greater tonnages and more ventilation air together with Rock Mechanics dictates now gave rise to the advent of the circular shaft and moreover to twin circular shaft systems with one shaft dedicated to ventilation requirements only and the other for hoisting rock, men, materials, services and downcast air.

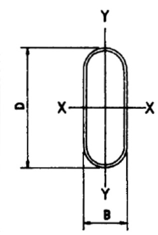
c) Circular Concrete Lined, Steel Equipped Shafts







SIZE D x B	WEIGHT PER FOOT	MOMENTS OF INERTIA X – X	Y – Y	MODULI OF SECTION X – X	Y – Y	SIZE DxB	WT/FT	MOMENTS OF INERTIA X–X	Y–Y	MODULI OF SECTION X–X	Y–Y	SIZE D x B	WEIGHT PER FOOT	MOMENTS OF INERTIA X – X	Y – Y	MODULI OF SECTION X – X	Y – Y
8"x 6"	35	115.06	19.54	28.76	6.51							8"x 6"	27.3	56.1	37.8	14.2	12.6
10"x 6"	40	204.80	21.76	36.11	7.25							10"x 6"	32.3	104.27	49.6	20.85	16.5
12"x 6"	44	316.76	22.12	52.79	7.37							12"x 6"	37.4	154.6	61.4	28.4	20.5
14"x 6"	46	442.57	21.46	63.22	7.15	14½" x 4"	41	215	24	30		14"x 6"	44.5	257.16	73.2	36.7	24.7
16"x 6"	50	618.09	22.47	68.88	7.49	16½" x 4"	46	314		38		16"x 6"	47.6	367.1	95.0	45.88	28.4
18"x 6"	55	841.76	23.64	93.53	7.88							18"x 6"	52.7	515.26	96.8	57.9	32.2
20"x 6"	65	1226.17	32.56	122.62	10.02	19½" x 4"	53	502	33	53		20"x 6"	57.9	688.6	108.6	68.86	36.2



COMPARATIVE TABLE OF PROPERTIES

COMPARATIIVE TABLE OF PROPERTIES (Figure 2)

i) Rolled Steel Section Buntons and Guides

The early circular shafts were equipped with rolled steel joist buntons and rolled steel channel guides. The buntons were usually 12"× 6" R.S.J. spaced, at centres that varied in time between 7'–6", 10'–0" and 15'–0". In view of the increased demand for ventilation air for increasing depths, the design engineers' efforts were directed to reduction in the amount of steelwork in the shaft. This was first achieved by changing the vertical pitches of the sets from 7'–6" centres to finally 15'–0" centres. Continuing efforts in this area resulted in the development of the streamlined or aerofoil bunton. (See figure 2).

ii) Diamond Aerofoil & Flattened Pipe Steel Buntons

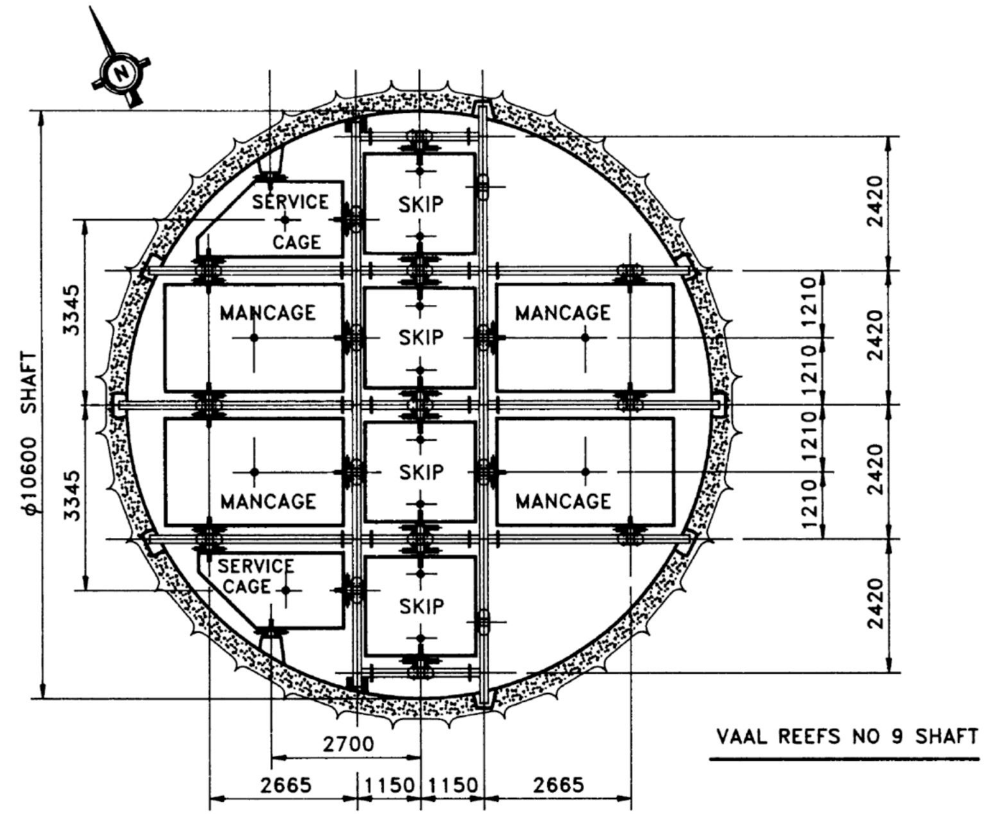
The development of the aerofoil section and later the flattened pipe section has been very adequately covered in previous papers of this nature and in particular, reference is made to the paper "The equipment of vertical shafts, Present South African mining practice and Development trends" by D.M.Bently.[1] These sections were initially successful in that they increased the airflow in the shaft over conventional buntons by 6%. However, latent stresses were built into both of these sectionsduring their manufacture.

These stresses developed when the plates were pressed and welded to form the sections. Although great care was taken to place the welds on the neutral axis, they were on the neutral axis in one plane only, i.e. the horizontal plane but not the vertical plane. These latent stresses and other factors would effectively reduce the fatique life of the buntons as was dramatically illustrated by occurrences at the President Steyn No 4 shaft, covered in the paper "The Shaft Steelwork Problem at No 4 Shaft President Steyn Gold Mining Company Limited by F.J.J.Blaauw,[2] where the steelwork failed completely due to fatigue and welding factors..

Efforts were intensified to find or design the bunton which would satisfy all conditions of strength requirements, aerodynamic properties and be free of built in stresses. These efforts led to the development of the hot rolled hollow section bunton in popular use today.

d) Current Deep Shaft Configurations

The hot formed hollow section bunton was used for the first time in a South African shaft at Vaal Reefs No 9 Main Shaft. (See Figure. 3) Commissioned in October 1983



(Figure 3)

i) Vaal Reefs No 9 Main Shaft

Shaft:	10600 diameter 2340 metres deep.
	Designed hoisting tonnage 330000 tons per month.
Skips:	2×25 ton payload hoisted at 16 metres per second. 2×22 ton payload, hoisted at 15 metres per second.
cages:	second. 4×3 deck mancages, 45 men per deck. Hoisting speed 15 metres per second.
Service	
Cages:	2×2 deck mancages, 22 men per deck. Hoisting speed 12 metres per second.
Guides:	320×150×71 Kg/m top hat section : Skips.
	308×102×57 Kg/m top hat section : cages.
Buntons;	500×150×10×89,48 Kg/m hot rolled hollow section.
	300×150×10×58,08 Kg/m hot rolled hollow section. Bunton sets are at 4500 centres.

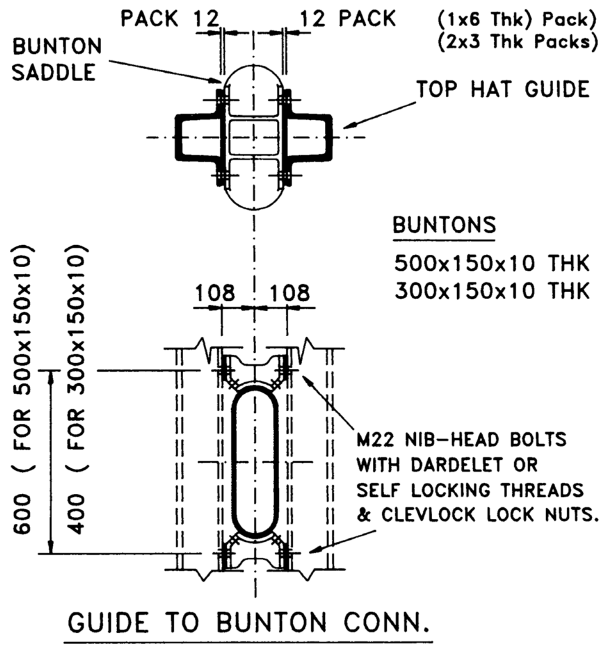
The hollow section shape combines good aerodynamic properties with efficient structural properties providing maximum resistance to vertical and horizontal bending and torsional stresses. The material conforms to the requirements of B.S.S. 4360=1979 grade 43 C with a minimum yield strength of 275 mpa. The grade 43 C was selected for its enhanced impact values.

The hollow section buntons are hot formed from hollows with one electric resistance weld seam. No weld material is introduced; the strip edges are fused together under roll pressure by the electric weld resistance (E.R.W.) process by means of induction. The outside weldbead is trimmed immediately after the seam is formed.

The E.R.W. process gives a weld seam strength equivalent to that of the parent metal and an effective 100% penetration. The subsequent hot forming into the bunton sections ensures a homogeneous section throughout and that residual stresses will be minimal. Should galvanising be specified as a means of corrosion protection, the process should be trouble free as the “innocent” bunton will not be subject to stress relieving in the galvanising bath.

ii) Guide to Bunton Connections

It is desirable that the guide be connected top and bottom of the bunton to reduce torsion and to ensure even deflections. The top and bottom connections effectively stiffen the guide over the area of the joint.



(Figure 4)

Earlier connections were made at the top of buntons only. The top hat guide is bolted to the bunton saddles using countersunk M22 diameter nib head bolts with dardelet or self locking threads and clevelock lock nuts. Behind the guide is a 12 mm laminated pack for adjustment which will be discussed under the installation chapter. The bolts are hot dip galvanised. (See Figure 4).

iii) Cast Steel Saddles

Because of the great number of saddles required for a major deep shaft, (20520 were required for Vaal Reefs No 9 shaft) it was decided to cast the saddles. The steel casting would have to be compatible to welding to the grade 43C bunton, be acceptable to the galvanising process and be able to withstand the impact and fatigue loading. The material specifications for the casting is :

The castings shall be cast in accordance with BS 3100–1976 Grade A1 but with the following restrictions :

Carbon	Less than 0,25%
Phosphorous	Less than 0,25%
Manganese	Less than 1,35%
Silicon	Between 0,15% and 0,30%

Charpy V notch impact value 27j at −15° C.
(See Figure 5).

iv) Bunton to bunton connections

These connections are effected by profile welding through the main bunton smaller hollow section of such length to allow access to huck bolt tools. The smaller section is then flanged for a single or double sided connection. The connection is made during the installation procedure. (See Figure 6)

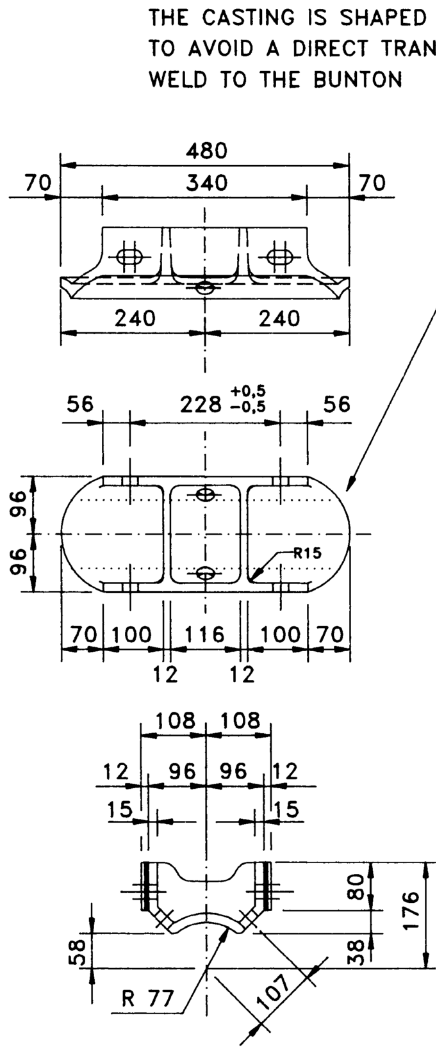
v) Alternative bunton to bunton connection

An alternative connection to the one described above, is to make use of the cast saddles used for connecting the guides. (See Figure 7).

Although great care is taken in the shaping of the saddles to avoid transverse welds to the bunton concern is still expressed at the welding required to effect both types of joints.

vi) Bunton to sidewall fixing

a) Conventional grouted pocket



CAST STEEL SADDLE

(Figure 5)

The open pocket is formed during the shaft sinking and lining process by means of window type pocket formers which are wedged to the shaft shutter. During the steel installation or shaft equipping phase, the bunton is packed level, wedged and grouted. (See Figure 8)

b) Positive return grouted pocket

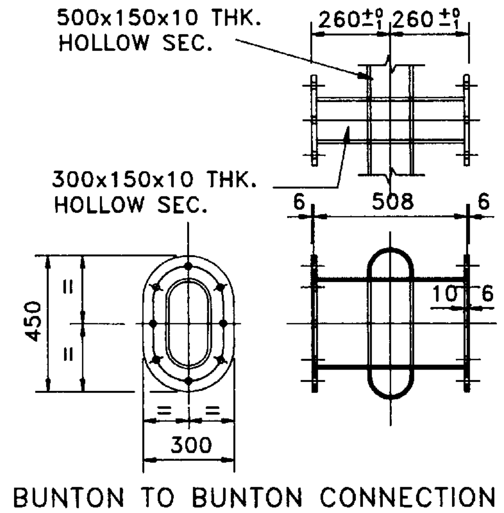
It is recognised that during the shaft steelwork installation process the grout will shrink away from the formed pocket in the case of the conventional and the positive return pocket. Deflections of the buntons will not be influended by the conventional pocket but will be restricted by the amount of shrink The amount of shrink may be controlled by the addition of aluminium dust to the grout. (See figure 8).

c) Open pocket

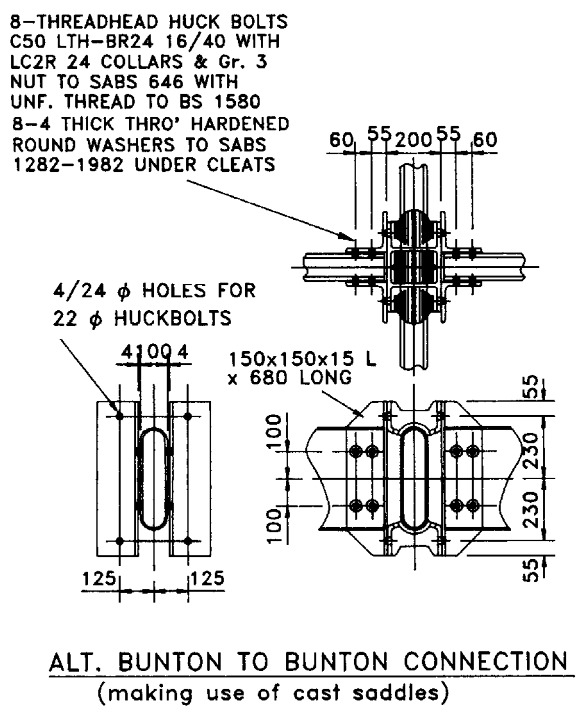
In this type of bunton fixing the steel pocket former is bolted to the shaft shutter and is cast into the shaft lining. The pocket remains open and the bunton is levelled, packed and bolted on final installation. (See figure 9a).

d) Bunton Ring

Large heavily constructed steel rings sometimes occupying half of the shaft circumference are attached to the shaft shutter and cast into the lining. The rings are pre-drilled to accept the bunton fixing brackets which are bolted with T head bolts. The bracket and the bunton are adjusted, set and bolted on final installation. Pipes and cables are also fixed to the ring. (See figure 9b).



(Figure 6)



(making use of cast saddies) (Figure 7)

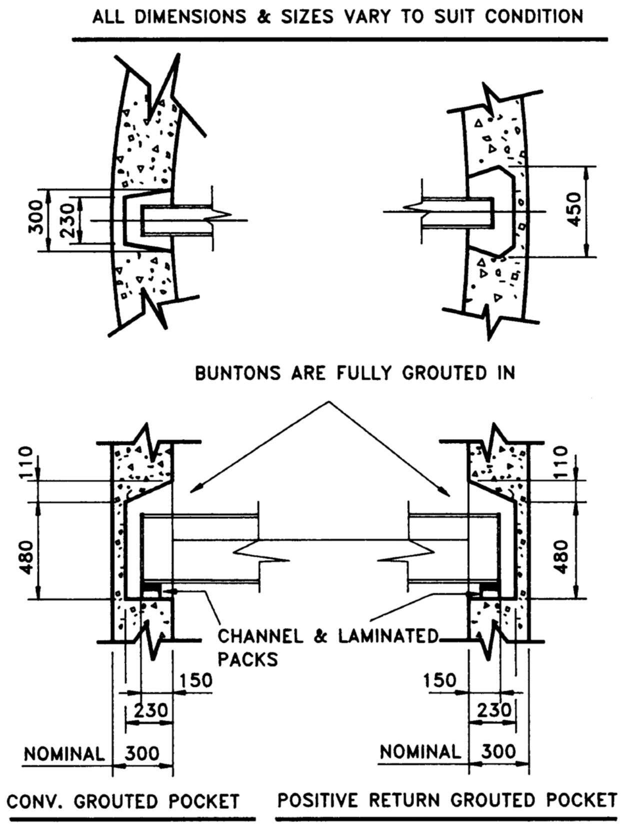
e) Guides

In keeping with the philosophy of designing shafts with stiff guides and flexible buntons, the top hat guide section has advanced from a 100 deep to a 200 deep section. (See figure 10). The increased range of guides available allows a better design selection to suit the different shaft parameters. Hot rolled hollow section guides have been considered and are in fact in extensive use with some mining houses whilst others prefer the top hat section because of its simpler connection details.

2.
DESIGN CRITERIA FOR NEW SHAFTS

a) Preamble:

Early shaft designs were based on designing the individual bunton as a unit with no thought given to the grid or frame of which it formed part. The advent of computers enabled an accurate analysis to be made of all the moments, direct forces and shear forces present in the grid system.



(Figure 8)

b) Design Philosophy

From experience gained in deep, high speed high rope end load shaft applications and numerous decelerometer tests performed the philosophy for rigid guides on flexible buntons has evolved. The system shall be designed so the horizontal bunton deflections should as near as possible be equal to the guide deflections.

c) Design parameters and loadings

The shaft layout, hoisting duty cycles conveyance sizes and payloads will all have been determined before shaft steelwork analysis can begin.

i) Life of mine or shaft

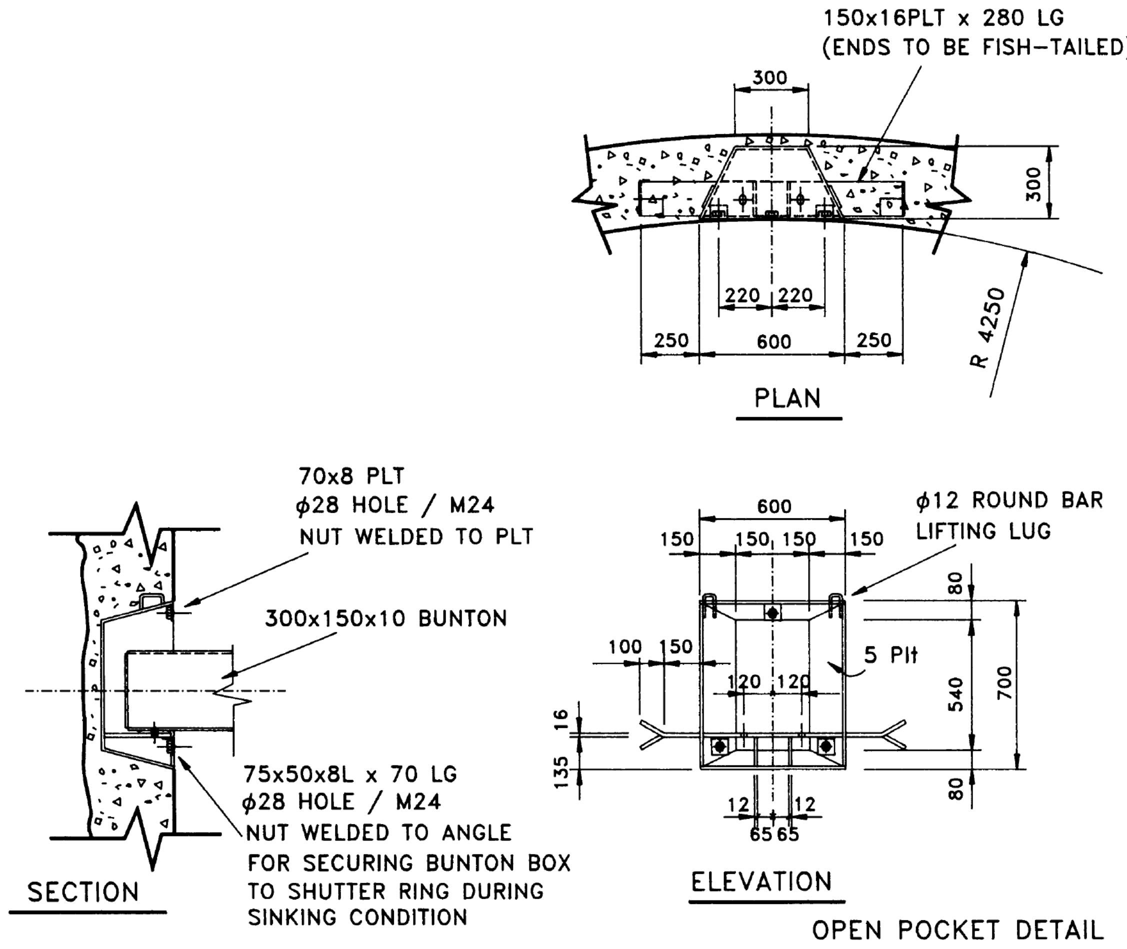
This is an important parameter which will determine the number of cycles the shaft steelwork will be subject to for fatigue considerations: a typical cycle calculation is:

Life of mine: 30 years.
Number of working days/year:
26×12=312
Number of working hours/day=22
Number of skip/cage trips/hour: 16
… Number of cycles=
30×312×22×16=3 294 720

ii) Loadings

The loads that are to be applied to both buntons and guides are :

a) Gravity loads.
b) Lateral forces from the conveyance rollers and slippers which are mainly caused by misalignment and slamming forces.
c) Vertical friction forces imparted by the rollers and the slippers.



(Figure 9a)

d) Buffeting loads.

These loads can vary between 5 and 20% of the rope end load for a typical well aligned shaft. Computer aided design analysis programmes which recognise the dynamic behaviour of shaft conveyances and the respose in the buntons and guides have been developed by S.D.R.C. of San Diego California (Slam) and Professor G.J.Krige of the University of the Witwatersrand (Discs) to design the shaft steelwork and the guides.

Two design cases must be catered for, namely

1) Allowable stresses and deflections are not to be exceeded;
2) Stresses governing fatigue life are not to be exceeded.

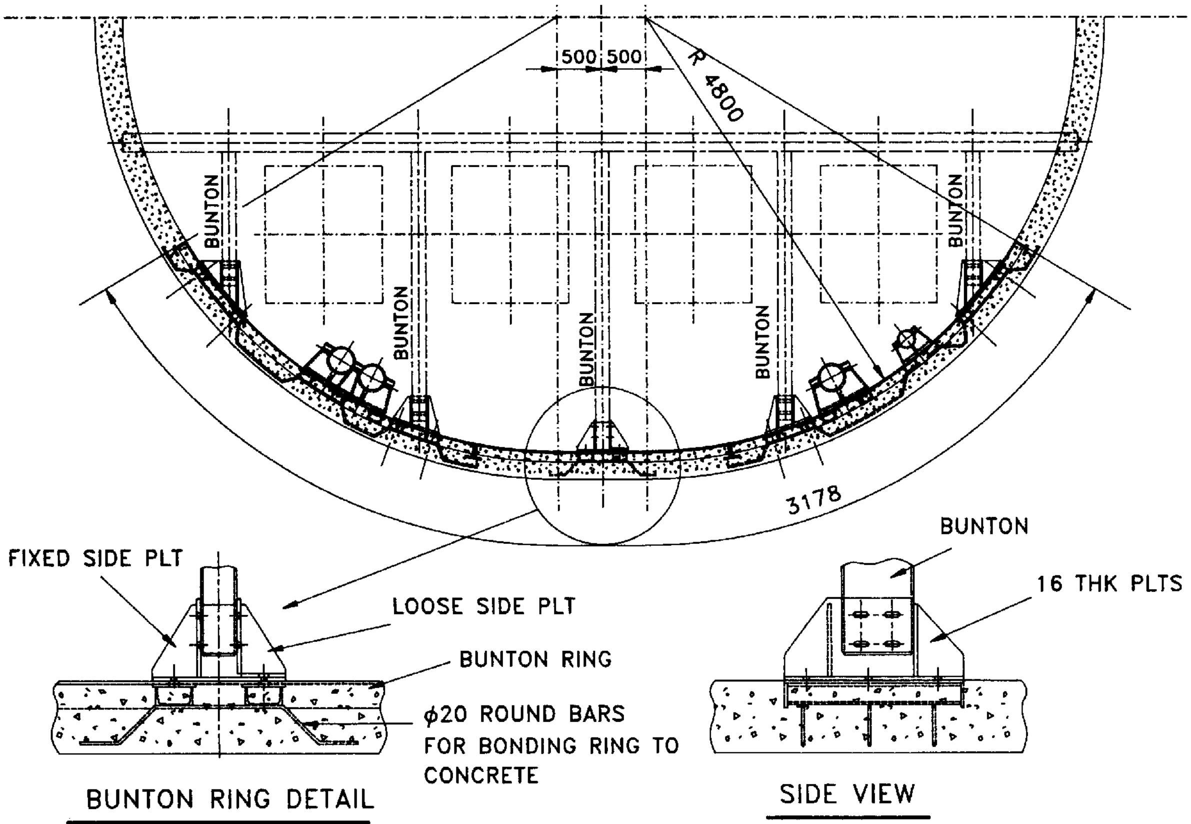
iii) Guide Rollers and Conveyance Slippers

The rubber tyred guide rollers fitted top and bottom of the conveyance at the guide positions are spring loaded to the face of the guide to a typical stiffness of 500 Kn/metre. The gap between the conveyance slippers or wearing liners and the guide is typically 10 mm. The load therefore to which the guide rollers will be subject, and the load they will impart to the guide is therefore 500 Kn/metre×0,01 metre=5 Kn before the roller will bottom out or dampen and the slipper will contact the guide.

The slipper loads as the conveyance "slams" from one guide to the other are computer analysed as described in the preceding paragraph. (See Figure 11).

iv) Buffeting of Passing Conveyances

The effect on the guides and shaft steelwork of large conveyances passing each other or of one conveyance passing a stationary conveyance at speeds of 16 metres/second in conjunction with downcast ventilation air travelling at speeds of 10 metres/second has been measured by research done by the South African Council for Scientific and Industrial Research.[3]



(Figure 9b)

Conveyances in close proximity to each other for example, cages on a double drum hoist, will pass each other in one specific identified area in the shaft and the steelwork in that area will be designed to resist buffeting loads which can be as high as 80% of normally applied slipper loads.

d) Corrosion Protection

Considering the aggressive atmosphere in most downcast production shafts and the fact that some shafts have predicted life spans of thirty years and beyond, corrosion protection for the shaft steelwork which must survive these elements becomes a prime consideration.

Properties of the mine water with regard to Ph. sulphate, chloride, nitrate sodium, potassium and copper contents and the velocity of the ventilation air must be established in order to specify the compatible corrosion protection.

The two most common protection means are hot dip galvanising and epoxy based painting. It has been considered in particularly aggressive areas of the shaft namely below the stations and below the loading pocket, to institute a "duplex" system namely to epoxy over hot dip galvanised steelwork.

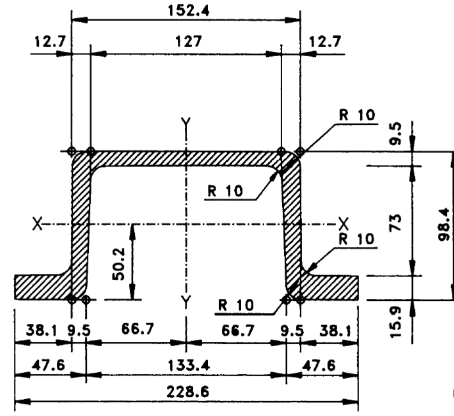
e) Acoustic Resonance

The velocity of the downcast ventilation air acting on a particular arrangement and type of bunton can create an acoustic resonance in the shaft of the type experienced at the Kloof Mine No 1 Shaft covered by the C.S.I.R. Contract report ME 1917[4] wherein it states that the design of new mine shaft installations should be scrutinised to avoid an acoustical resonance, as follows : The frequency of the standing transverse half wave mode in the shaft should be calculated from $f = \frac{C}{2W}$ where W=0, 88×shaft diameter and C is the speed of sound at the expected ambient temperature of the air. Next the frequency of the Vortex shedding from any transverse shaft buntons should be calculated from $n = \frac{StV}{D}$

The Strouhal number for the bunton shape must be known and the bunton thickness D and air velocity V are usually design parameters fixed for a particular shaft.

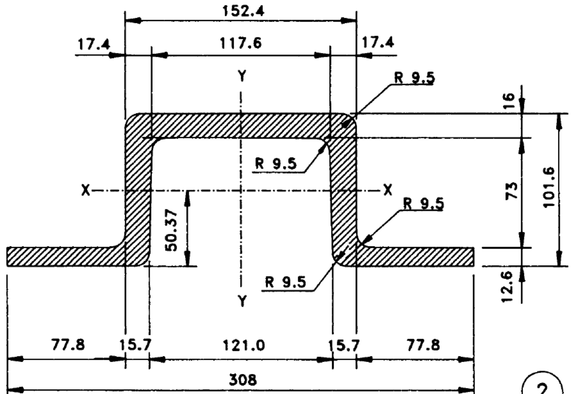
The Strouhal number: when vortices are shed from a prismatic body of any cross sectional shape in a crosswise air stream, the so called Strouhal number pertaining to the cross section's shape is of great importance. This non—dimensional number is defined as: $St = \frac{nD}{V}$

where n is the frequency at which vortices are shed from the body and D is a typical linear dimension usually the thickness of the cross section of shape perpendicular to the air velocity V over the prism.



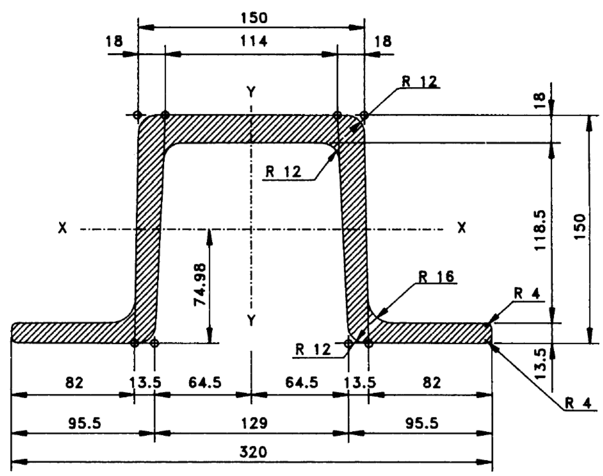
1

152.4 x 98.4 x 228.6 TOP HAT GUIDE SECTION

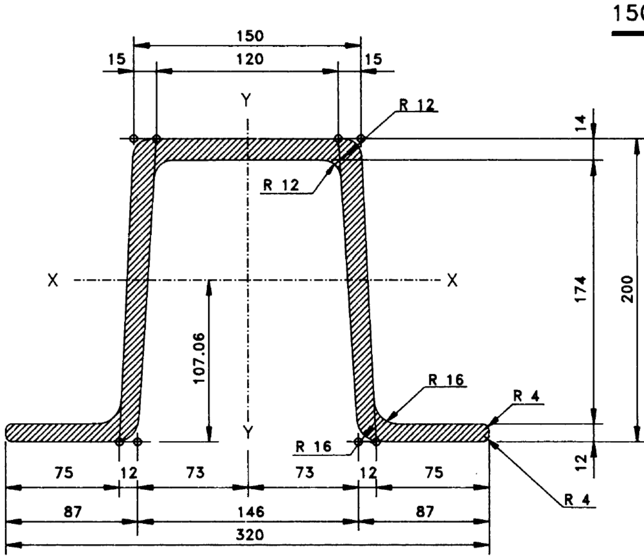


2

152 x 102 x 308 TOP HAT GUIDE SECTION



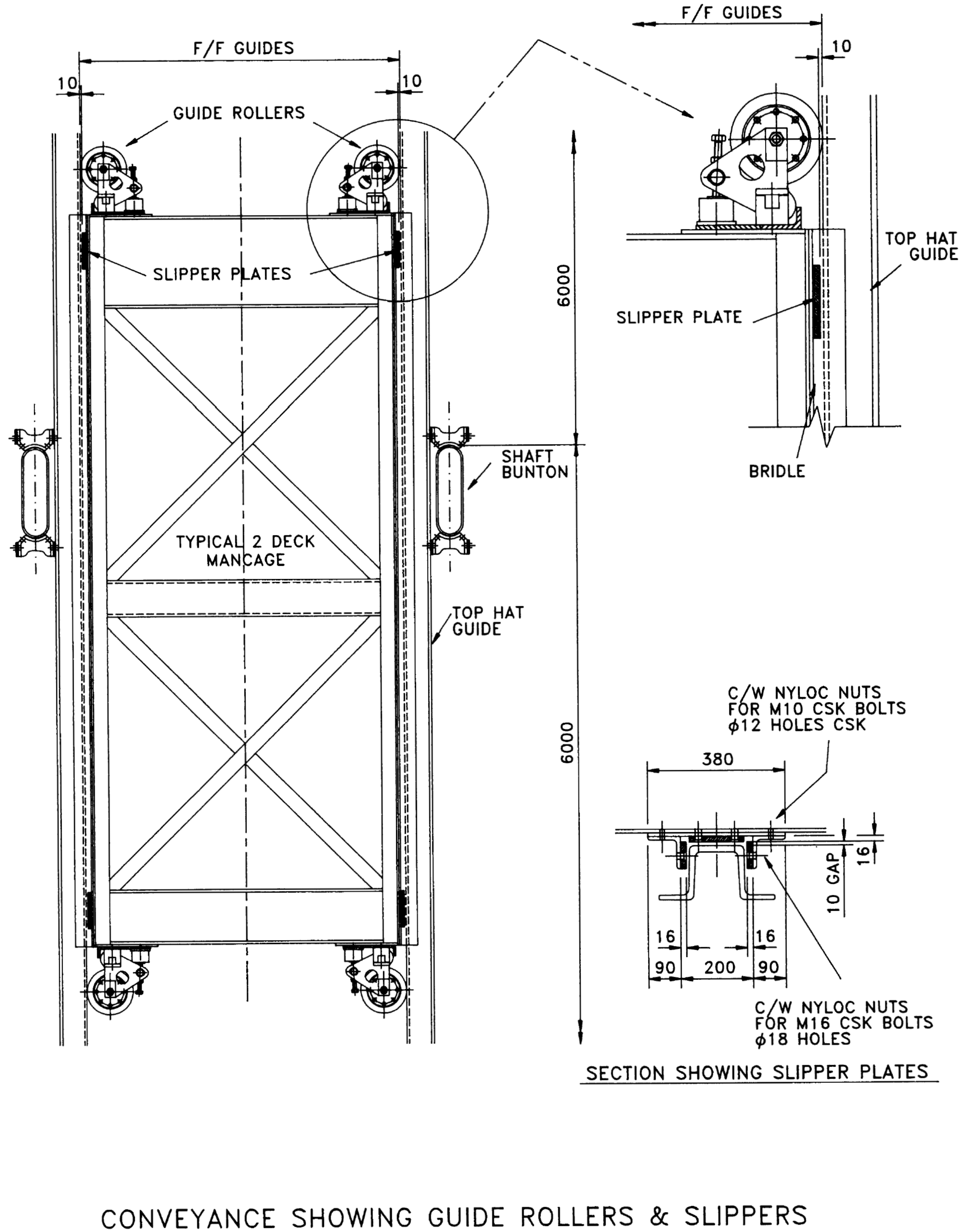
150 x 150 x 320 TOP HAT GUIDE SECTION 3



150 x 200 x 320 TOP HAT GUIDE SECTION 4

GUIDE MATERIAL SHALL CONFORM
TO BS 4360–1972
WELDABLE STRUCTURAL STEELS
GRADE 43C (U.O.N.)

(Figure 10)



(Figure 11)

From equation (1) it can be seen that once the strouhal number for a beam of given cross section is known the frequency of vortex shedding at any given air speed over the beam can readily be calculated. Once these factors are known the shaft parameters may be varied sufficiently to prevent resonance from occurring.

3.
INSTALLATION

a) Preamble

The installation of the shaft steelwork is the final link in an elaborate chain of events through the specification, design, shaft sinking and lining, steelwork fabrication, steelwork jigging, surveying, installation and final commissioning. It needs to be emphasised here, that great care taken through all of these phases will culminate in a well aligned, trouble free shaft. This chapter is devoted to the steel installation and related aspects.

b) Steelwork and Shaft Tolerances

i) Rolling and fabrication tolerances

a) Buntons:

The rolling tolerances are: Straightness: to be 0,10% measured at the centre of the length produced.

Length Tolerance: ±3 mm.

Depth and Width: a tolerance of +1 mm is required on the width as well as the depth of the section. The individual buntons will be fabricated in jigs to maintain the above tolerances. If the buntons are galvanised, the same final tolerances will apply :

b) Guides:

The top hat guide tolerances are:

Width over flanges ±4 mm.

Width of head: +1,5 mm–0,8 mm

Height: +0,8 mm–0,6 mm

Camber: 3 mm per 3 000 mm length or 0,10%

Length: The guides are ordered at+25 mm− 0,0 but after drilling grinding and matching, are supplied at +0,0 −2,0 mm

ii) Shaft Tolerances

a) Shaft Lining Tolerances

The shaft shutter or formwork shall be so designed that the tolerance on radius, diameter and circumference at any given point in the shaft will be ± 0,20%.

b) Shaft Verticality

The shaft shall be sunk and lined to maintain a vertical tolerance of 25 mm per 300 metres, and shall be so corrected that the maximum deviation from vertical over the entire length of shaft shall not exceed 35 mm.

c) Shaft Steelwork Installation Tolerances

The design of the shaft steelwork shall allow for finite adjustment in bunton and guide fixings to maintain the correct guide gauge and alignment to the following tolerances:

Guide gauge ± 0,10%

Guide verticality: ±3 mm per 300 metres or 0,001%.

d) Guides

Theory and practice indicate that the greatest shock loading to a guide string, occurs at the guide joint due to a protrusion[5] therefore great care is taken to match the guides. A typical matching system is:

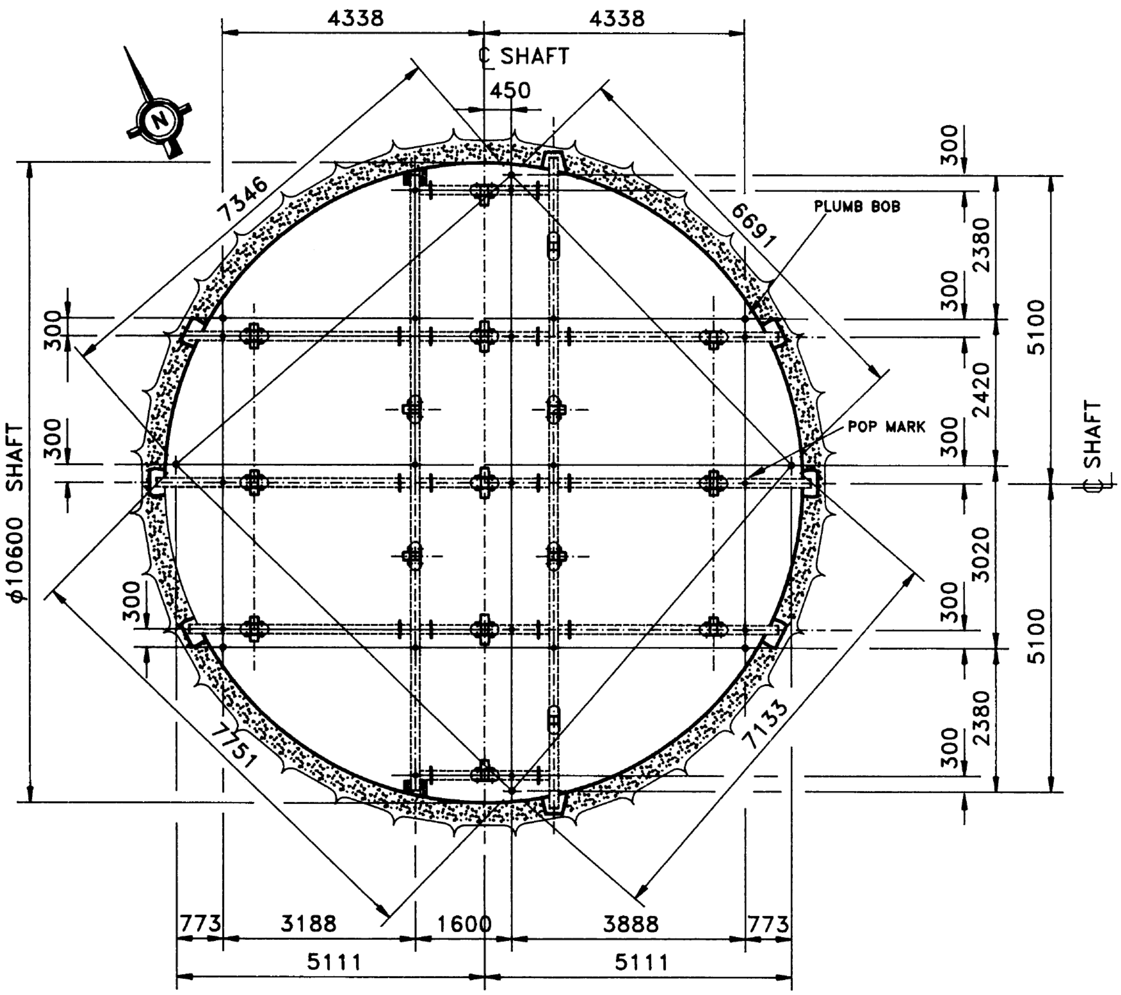
The guides are to be classified and colour banded in three sizes, as follows

Size 1: Zero to 1 mm undersize banded yellow

Size 2: Zero to 1 mm oversize banded red

Size 3: +1 mm to +1,5 mm oversize banded white

Transition guides between sizes 1 and 2 banded half yellow and half red. Between sizes 2 and 3 are banded half red and half white.



VAAL REEFS NO 9 SHAFT

PLUMB LINE & POP MARK SURVEY PLAN

(Figure 12)

The checking and matching of the guides must be done at normal workshop temperature after the guides have been under cover for a minimum period of 24 hours.

e) Manufacturing and assembly jigs

As stated earlier the individual buntons will be fabricated and checked in workshop jigs. The fabricator will provide an assembly jig which will duplicate the shaft guide positions and plumb bob line positions exactly. After galvanising each bunton set will pass through the fabricators assembly jig.

At the shaft collar an assembly jig which again duplicates exactly the shaft guide positions and the plumb bob lines the steel is checked and hard marked with reference to plumb line positions. The exact guide gauge is established and the correct laminated packs behind the guides will be installed in the shaft with each particular bunton set.

f) Surveying and dimensional checks

The position of plumb bob lines in the shaft and the corresponding centre punch or pop marks on the buntons must be designed to facilitate the correct installation of the shaft steelwork. An example is illustrated in figure 12.

g) Installation or equipping methods

The shaft steelwork is installed in the shaft in basically two manners, either from the top down or in shafts that have a brattice wall, from the bottom up. The more conventional method of equipping from the top down with variations is described as follows: After the shaft sinking and lining has been completed, the multi deck sinking stage is modified for stripping and equipping. These modifications consist of removal of the lashing gear and one or two decks of the normally five decked stage. The stage is originally designed with these modifications in mind.

The temporary shaft sinking services are then stripped on the way up. This is also an opportune time to instal the permanent shaft services such as refrigerated water columns and compressed air columns which are then installed on the way up.

When all the temporary services have been stripped out and if applicable the permanent services installed, the stage will be at the shaft collar and the headgear changeover from sinking to equipping conditions can be done. The guide equipping conveyances are installed and buntons and guide installation/equipping can begin.

The modus operandi for method 1 as shown in figure 13 is: the buntons are installed from the top deck of the stage and the guide installation follows behind from the guide equipping conveyances.

The disadvantages of method 1 are that some people are installing buntons from the stage and others are installing guides from the equipping conveyances after the buntons have been installed.

Current shaft steelwork equipping trends are illustrated in method 2, figure 13.

Considering the fact that the guides are the most important items to be aligned and the buntons play a supportive role, a method was devised to install the guides first and the buntons after. The installation procedure for method 2 is: The guides are brought down by the guide conveyances and they are lowered through specific openings in the stage. The guide is then connected to the previously installed guide. The conveyance departs and brings down the buntons which are installed from the top deck of the stage. The surveying, checking, and guide gauge checking, as described before, is then carried out and the buntons are levelled and set to the guides. All of this work is done from the top deck of the stage. Other methods of equipping as well as pipe and electric cable installation are not discussed.

h) Shaft Commissioning

After all buntons and guides have been installed, the guide installation conveyances are removed and the stage is dismantled at the shaft bottom and removed.

The permanent shaft conveyances are installed and decelerometers are attached. Each compartment of the shaft is now thoroughly examined at slow speed, approximately 40 metres per minute and the horizontal accelerations of the conveyance are measured by the decelerometer.

Decelerometer testing is a technique in which an accelerometer, commonly known as a decelerometer, is used for measuring conveyance acceleration/deceleration in a lateral plane. Its purpose is to determine the severity of shock loads produced by misaligned shaft guides. Testing is performed by mounting the instrument on the conveyance and operating the conveyance, empty and loaded, at various speeds up to and including full speed.[6]

Adjustments and corrections, if necessary, are made and the shaft then becomes fully operational.

4) CONCLUSION

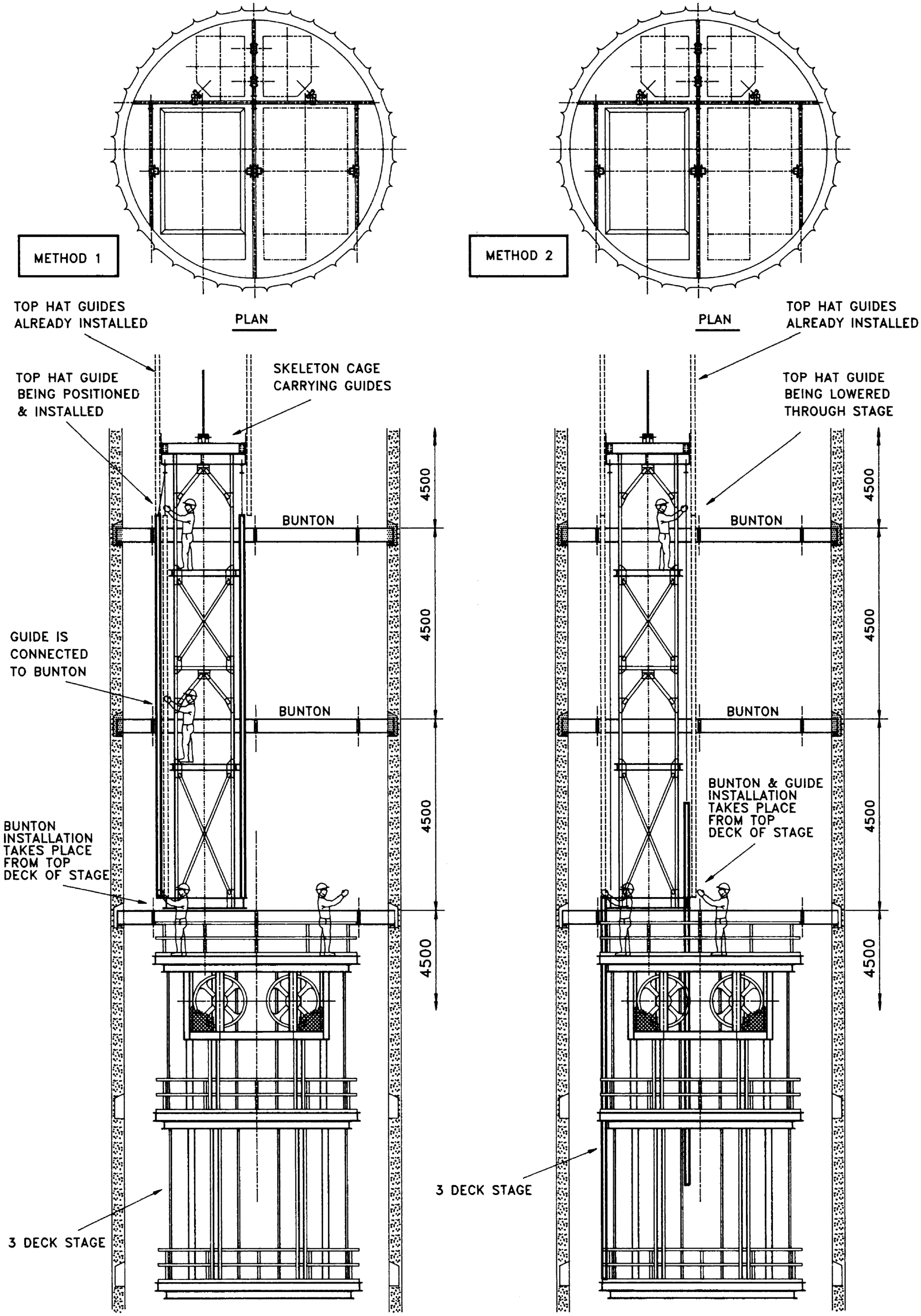
This paper has attempted to illustrate the development of shaft steelwork trends from the early South African mining days to where we are today with our large deep high production shafts. The Chamber of Mines of South Africa who instituted research programmes to study the dynamics of shaft conveyances and steelwork is to be credited for a lot of the development that has taken place.

In the chapter relating to installation, emphasis was laid upon tolerances in an effort to illustrate that with the correct mechanisms in place in the programme and adherence to quality control, excellent shafts are being and will continue to be created.

Having discussed the development trends to date what are future trends likely to be?

Hoisting speeds, currently at 16 metres per second are likely to be increased to 18 and even 20 metres per second. With the prevalence today of shaft pillar mining and the rock mechanics effect on the barrel of the shaft, it is expected of the shaft lining, shaft steelwork and guide designs to accommodate shaft closures of the magnitude of 500 mm and horizontal displacements to 75 mm in known areas of the shaft.

Integrated hoisting systems possibly utilising linear motor technology are also being investigated, whichever way trends develop, we trust the industry will meet the challenge.


METHOD 1
METHOD 2
PLAN
PLAN
TOP HAT GUIDES ALREADY INSTALLED
TOP HAT GUIDE BEING POSITIONED & INSTALLED
SKELETON CAGE CARRYING GUIDES
BUNTON
BUNTON
GUIDE IS CONNECTED TO BUNTON
BUNTON INSTALLATION TAKES PLACE FROM TOP DECK OF STAGE
3 DECK STAGE
4500
4500
4500
4500
TOP HAT GUIDES ALREADY INSTALLED
TOP HAT GUIDE BEING LOWERED THROUGH STAGE
BUNTON
BUNTON
BUNTON & GUIDE INSTALLATION TAKES PLACE FROM TOP DECK OF STAGE
3 DECK STAGE
4500
4500
4500
4500

(Figure 13)

5.
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6.
REFERENCES

1) The Equipment of Vertical Shafts. Present South African Mining Practice and Development by D.M.Bentley;

2) The Shaft Steelwork Problem at No 4 Shaft President Steyn Gold Mining Company Limited by F.J.J.Blaauw.

3) Aerodynamic Buffeting Forces Between Large Mine Shaft Conveyances Travelling at High Speeds, C.S.I.R.Contract Report ME 1897.

4 The Kloof No 1 Shaft Noise Problem, C.S.I.R. Contract Report ME1719.

5) Towards a Better Understanding of Mine Shaft Guides, by J.S.Redpath & W.M.Shaver.

6) The Value of Decelerometer Testing by Keith Jones and Largo Albert.

Design, implementation and monitoring of full-face blasts to extend a shaft at Atomic Energy of Canada Ltd.'s underground research laboratory

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A 255 m-deep vertical rectangular shaft was deepened by blasting full-face rounds. The shaft extension had a circular cross section of 4.6 m diameter. Rounds were based upon an off-centre burn cut, the position of which was alternated for successive rounds. The cut and adjacent area were designed to minimize Instances of sympathetic detonation and dynamic pressure desensitization. Damage to the wall of the shaft was minimized by using

1. highly decoupled charges in closely spaced perimeter blastholes, and
2. moderately decoupled charges in blastholes in the penultimate ring.

The Initial blast design was improved by incorporating the results of the Canadian Industries Limited (C-I-L) SABREX blast model. Sophisticated blast monitoring equipment was used to detect

1. variability of the firing times of the long-period delay detonators; and
2. instances of overlap, crowding, sympathetic detonation, dynamic pressure desensitization, robbed burdens and misfires.

The rationale for the initial blast design and the design defects indicated by blast modelling and monitoring are explained. When 3.5-m-deep rounds were fired, an advance of 3.3 m and an average of 29% of perimeter blasthole traces were achieved.

INTRODUCTION

Atomic Energy of Canada Limited (AECL) is constructing an Underground Research Laboratory (URL) in a granite batholith near Lac du Bonnet, Manitoba. The URL facilities will provide a representative geological environment for a variety of geoscience research and development programs associated with the Canadian Nuclear Fuel Waste Management Program. This program will assess the concept of used nuclear fuel disposal deep in the plutonic rock of the Precambrian Shield.[1]

In 1984 and 1985, AECL sank a 2.8 m×4.9 m rectangular shaft to a depth of 255 m. This shaft was excavated by conventional bench-type blasts. Between 1987 July and 1988 June, the shaft was deepened a further 188 m. The shaft extension has a circular cross section, the design diameter being 4.6 m. The design axes of the rectangular and circular parts of the shaft are coincident.

The circular section was blasted full face to fulfil AECL's research requirements. Excavation was carried out during two shifts, afternoon and night, seven days per week. Day shift was reserved for research activities. The aim was to excavate one round each day. By 8 a.m. each day, the shaft bottom had to be mucked out, the shaft wall scaled, and the bottom and wall washed to structural concrete emplacement standards to facilitate geological mapping. Sinking operations were halted several times, for periods of up to one month, while geomechanical and hydrogeological instruments were installed in the shaft at instruments arrays.

OBJECTIVES

The objective of the blast design was to develop and implement a blast round that would break a high percentage of the drilled depth and would give well fragmented and loose muck. A second priority was to develop and use a controlled blasting technique to minimize excavation damage to the shaft wall. There was also a need to control flyrock and to optimize the blast design over the minimum sinking distance. Because advance rate was not the principal priority, the work was somewhat different from most mining and civil engineering projects.

RELEVANT GEOLOGY

The characteristics of the granite varied appreciably. Table 1 shows the geotechnical properties of the Lac du Bonnet granite at the URL.[2] The rock had high strengths and almost no natural fractures. Accordingly, blasts were required to create essentially all of the fragmentation. The high rock quality mitigates good fragmentation and minimizes overbreak.

Table 1 Standard geotechnical properties of Lac du Bonnet granite

Type of Granite	Pink	Grey
Uniaxial Compressive Strength (MPA)		
Range	134–248	147–198
Mean	200	167
Young's Modulus (GPa)		
Range	53–86	46–64
Mean	69	55
Hoek & Brown Failure		
Parameters		
m	31.17	30.54
s	1	1

BLAST DESIGN CONSIDERATIONS

The shaft was extended using a full-face blasting method so

- controlled blasting could be used effectively to limit blast damage; and
- the optimum length round could be cycled each day during the two shifts available to the sinking crews (research crews occupied the shaft on day shift).

Also, the slightly concave face produced by full-face blasting was more desirable for the researchers who were developing models for predicting the rock mass behavior. Factors considered in the blast design Included cut design, initiation sequence and delay scatter, production or mid-ring blasthole design and the design of the perimeter blastholes.

Cut design

When a confined explosive charge detonates, explosion gases perform best where free faces (i.e., rock/air interfaces) exist. A free face that is parallel is far preferable to one that is normal to the axes of blastholes. Parallel free faces have the beneficial effects of

1. promoting fragmentation, and
2. allowing broken burdens to be displaced sufficiently to give loose muck.

In full-face shaft rounds, the initial free face (i.e., the shaft bottom) is normal to the blastholes and is therefore inadequate. This makes it necessary to develop a "cut" to the design depth in the face. The walls of the cut act as free faces to which later-firing charges can break. In full-face rounds in circular shafts, there are two principal types of cut: cone cuts and burn cuts.

Cone cuts

Specific drilling and powder factors are generally lower for cone-cut rounds than for burn-cut rounds. But cone cuts have major disadvantages, including the following:

1. Rock from the cut tends to be
   - poorly fragmented, especially in strong massive rocks; and
   - violently ejected, with a high potential to damage shaft installations and equipment.
2. Whenever efforts are made to pull deeper rounds, it is necessary to change the blast-hole pattern.

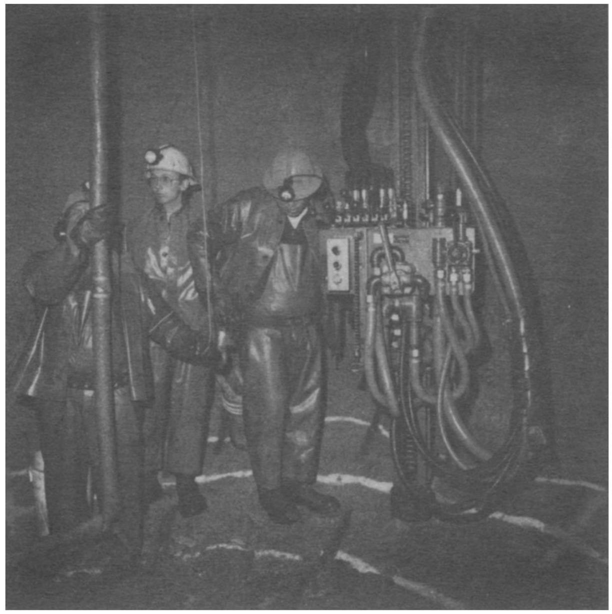


Fig. 1 Crews are shown setting up one of the two single boom drill jumbos used to drill the full-face rounds. The jumbos improved drilling accuracy and vertical hole alignment.

3. The tight spacing of the toes of cut blast-holes is often reduced by poor control over blasthole alignment. Therefore, the earliest-firing charge has a relatively high capability of either initiating (by sympathetic detonation) or desensitizing (by dynamic pressures) the adjacent charge(s).

Where sympathetic detonation[3] occurs, the intended timing is overridden. Charges detonate virtually simultaneously, causing undesirable increases in ground vibrations, overbreak potential and flyrock. Sympathetic detonation is most common when using the more sensitive nitro-glycerine-based explosives.

Where dynamic pressure desensitization[3] occurs, later-firing detonators in the cut can fail to initiate the densified and, hence, desensitized charges. Therefore, the probability of pulling the cut (and the rest of the round) to full depth decreases. Such desensitization is most probable in watergels and emulsions that do not contain high-strength glass microballoons.

Burn cuts

When a shaft round is based upon a burn cut, all holes should be as vertical as possible (see Fig. 1). Because blastholes are essentially normal to the initial free face, the degree of fixation of the earliest-firing charges is extremely high. This "tightness" is reduced by providing "relief" holes to which the cut blast-holes can break. When the radially expanding strain wave reaches a relief hole, wave reflection promotes fragmentation of the rock between the charge and relief hole. After fragmentation, relief holes provide the voids Into which the blast-generated rock fragments expand. As the diameter and/or number of relief holes Increases, so does the probability of pulling the cut to full depth.

In the cut, the bottom element of each charge can break to only the free faces provided by the adjacent relief holes. If the depths of blast-holes and relief holes are equal, the free faces for the bottom element of the charge are very limited. The ability of the bottom element of each cut charge to break cleanly to the relief holes can be increased by drilling the relief holes about 300 mm deeper than the blastholes. This has the effect of almost doubling the area of free face for the bottoms of earliest-firing charges.

Relief holes can be positioned to reduce the ability of the earliest-firing charge to either initiate or desensitize an adjacent later-firing charge. In the burn cut shown in Fig. 2, the four charges are relatively closely spaced.Also, charge A can "see" charges B and D and, therefore, the radiated compressive wave can impact these charges to the extent that either sympathetic detonation or dynamic pressure desensitization occurs. If B detonates, it can initiate or damage C. If C detonates, it can initiate or damage D (which could have been previously damaged by A). Where long-period delays are employed, radial cracks from blast-hole A can possibly propagate toward, and enter, B and/or D, allowing high-pressure gases to jet into these later-firing blastholes (especially D) and either desensitize or physically eject their charges.

In the cut shown in Fig. 3, the positions of relief holes are such that

1. blastholes are more widely spaced, and

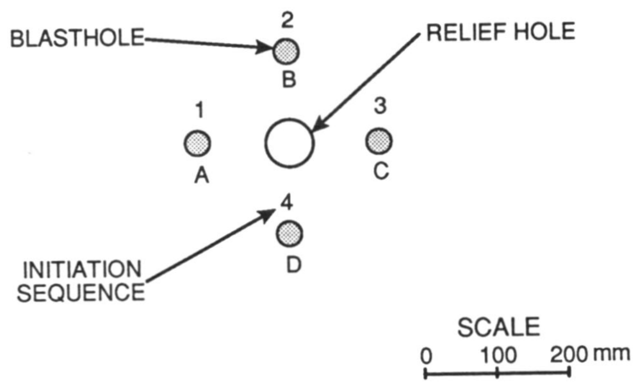


Fig. 2 Simple but unsuitable burn-cut configuration

2. none of the blastholes can “see” the other two blastholes.

The intensity of the strain wave (generated by A and experienced by B and C) is reduced as a result of

1. the greater attenuation (caused by greater divergence and absorption effects), and
2. the shielding ability of the relief holes.

By positioning relief holes between blastholes, the probability that cut charges will undergo the intended detonation sequence Increases. The “helper” charges around the cut (i.e., blast-holes D, E and F in Fig. 3) are not shielded from relief holes, they are at a sufficient distance from A, B and C to make the probability of sympathetic detonation or desensitization remote. but In a burn cut such as shown in Fig. 3, fragmentation is excellent and there is a lower tendency for rock to be violently ejected.

Mid-ring design

As initiation proceeds outwards from the cut, each succeeding charge has a free face of progressively larger area and, therefore, can break a greater burden distance. Eventually, free-face area and burden distance attain maximum values. As shaft wall is approached, the need to control overbreak encourages a reduction of effective charge diameter and burden the distance.

In circular shafts, it is necessary to locate blastholes on concentric rings. In the interests of maximizing fragmentation and muckpile looseness and limiting ground vibrations, the drilled blasthole spacing: burden ratio should lie in the approximate range 1.2–1.3.

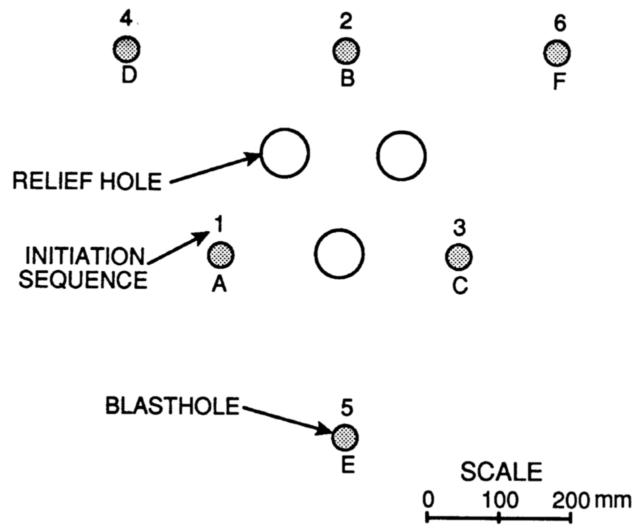


Fig. 3 Improved delay allocation for burn cut

For a given charge mass per delay, ground vibration intensity increases with

1. decreases in powder factor, spacing: burden ratio and available face area; and
2. increases in the mass of previously broken rock that chokes the face.

When the burden on a charge becomes excessive, the explosion gases are "contained" within the blasthole, generating abnormally high ground vibrations. In effect, the amount of energy conserved as a result of the reduced breakage and displacement is manifested as additional seismic energy.[4]

Perimeter design

Design of the spacing, burden, charge concentration, initiation sequence and delay timing of perimeter blastholes have an important effect on the amount of overbreak.

Perimeter blastholes should be as parallel as possible to the earlier-firing blastholes. Because perimeter blastholes must have a lookout angle (which should be minimized), the burden distance increases toward their toes. It is the toe burden rather than the collar burden distance that is important. If the toe burden is excessive, "bootlegs" will remain after mucking.

In each perimeter blasthole, the mean energy yield per metre of charge length should be low. Because blastholes are usually wet, it is generally necessary to employ decoupled charges of a water-resistant explosive. For a given blast hole diameter, progressively better wall quality can be achieved by using increasingly decoupled charges in blastholes on closer centres, the optimum combination of charge concentration and blasthole spacing being dictated largely by the time and cost of drilling. In practice, the blast designer has to select a charge concentration by using one of a limited number of available perimeter explosives. Therefore, freedom in selecting a charge concentration is restricted.

Because presplitting is, as yet, unsuccessful in shaft sinking, it is necessary to apply a smooth-wall blasting technique, in which the lightly charged perimeter blastholes detonate after all of the production blastholes. Ideally, all such perimeter blastholes should detonate simultaneously, but this is not yet possible, since all current delay detonators exhibit appreciable "scatter". This is especially so for the higher delay numbers (see section below). Because the scatter for a No. 18 long-period delay, for example, is likely to be at least ± 200 ms, the probability that perimeter charges will detonate with sufficient simultaneity to assist perimeter split formation is low. Therefore, it is preferable to use groups of, e.g., delay numbers 17 and 18 to reduce the superposition and, hence, resultant of individual vibration wave packets.

Initiation sequence

The initiation sequence and delay timing should be such that

1. one charge breaks and displaces its burden rock before a dependent or semi-dependent charge detonates,

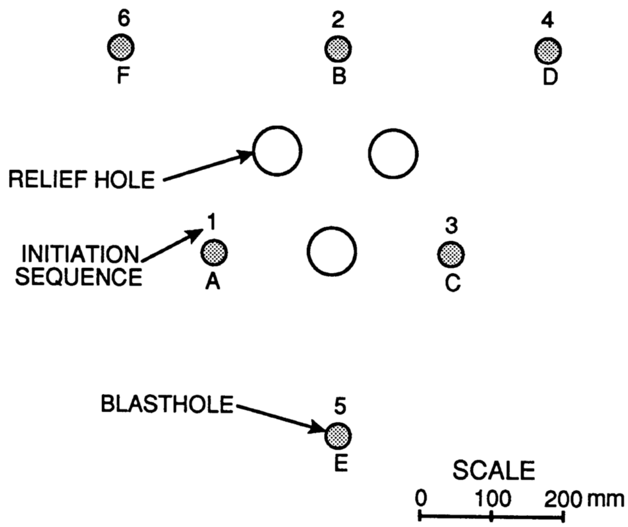


Fig. 4 Sub-optimum delay allocation for burn cut

2. broken burdens for two or more simultaneously initiated charges are not required to swell Into an Inadequate void volume (This is especially important where the burdens on cut blastholes can expand only into the limited void provided by the relief holes.), and
3. the ground vibration intensity during a blast is relatively uniform rather than a series of peaks and troughs.

Where long-period delay detonators are used in burn cuts, there is a tendency to place two or more charges on the same delay number. Operators who adopt this practice argue that the scatter for long-period delays is so great that the probability of simultaneous detonation is very low. The probability that charges on the same (e.g., No. 1) delay will compete for a void volume sufficient for only one of the charges is always unacceptably high. With long-period delays, therefore, cut charges should be initiated by consecutive delay numbers.

In the burn-cut configuration shown in Fig. 3, it is important to comprehend the reasoning for the delay allocation for charges D, E and F. The delays shown provide the longest minimum delay for dependent charges".

Fig. 4 shows a delay allocation in which extending the minimum delay between dependent charges has not been considered. In this case, D shoots only one delay after C. Therefore, if C were to fire late and D were to fire early, the period in which C is required to break and eject Its burden rock could possibly be inadequate. When selecting an initiation sequence around a cut, one should allocate the next delay to the blasthole that has been relieved by the longest minimum time interval.

Outside the area of the cut and helper charges, delays need to be allocated to meet the competing demands of maximum progressive relief of burden and minimum peak ground vibration.

Currently, the assimilation of these competing requirements takes the following Into account:

1. the increase in absolute scatter with delay number and, therefore, the opportunity to increase charge mass per delay with delay number (for a given vibration limit); and
2. the Increase in ground vibration intensity with the tightness of the charge. (The vibration produced by the earliest-firing charge in a burn cut, for example, is up to about five times higher than that generated by a charge of equal mass located well outside the cut.)

At present, all conventional delay detonators exhibit "scatter" (i.e., delay-time variability). As a percentage of nominal firing time, scatter is quite small, especially for detonators from the same batch. But in absolute terms, scatter becomes considerable for detonators having high delay numbers. Manufacturers often choose to Increase the nominal delay interval with delay number, to minimize the probabilities of "overlap" (which occurs when delay number n+1 detonates before delay number n) and "crowding" (which occurs when a late-firing delay number n 4–1 detonates so soon after an early-firing delay number n that the resulting interval is insufficient to allow the earlier-firing charge to create the free face needed by the dependent charge.) Being aware of the deleterious effects of possible overlap, blast designers often place dependent charges on alternate rather than consecutive delay numbers. The nominal firing Intervals for dependent charges are much greater than those needed for progressive relief of burden. If scatter could be eliminated and confidence in the firing times of detonators could be boosted to a very high level, blasts that currently last about eight seconds could be fired in about two seconds. In a full-face shaft round, this would have the beneficial effect of reducing the degree of choking of the faces to which later-firing charges

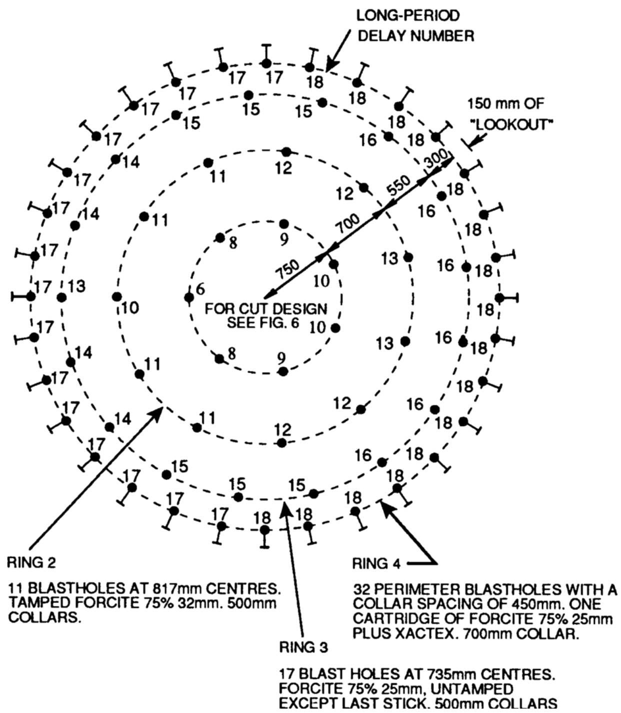


Fig. 5 Initial full-face blast design for rings 2 to 4

shoot, but without reducing the inter-charge delay to the stage at which the super-position of individual vibration wave packets becomes appreciable.

INITIAL BLAST DESIGN

The delay allocation for the URL shaft round design, shown in Figs. 5 and 6, has the following beneficial features:

1. one hole per delay and greatest assured relief in and near the cut,
2. a small number of dependent charges on consecutive delay numbers, and
3. a charge mass per delay that increases with increasing delay number (and scatter) and with decreasing tightness of charges.

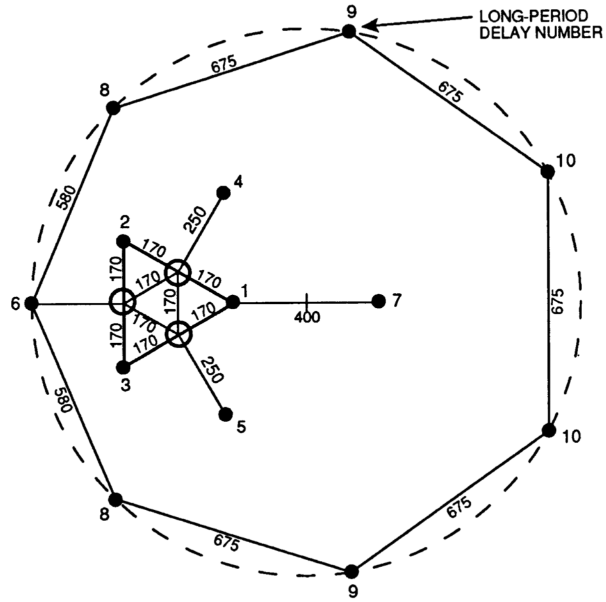
As the perimeter is approached along any radial line (see Fig. 5), dependent charges detonate on alternate delay numbers, thereby raising the probability of achieving good progressive relief to a very high level. Unfortunately, the long nominal delay times between dependent charges (delay times that could be considerably shorter if one could eliminate scatter) have a potential disadvantage, viz., as this delay Increases, the earlier-firing charge is more capable of

1. dislocating the adjacent charge; and
2. blowing, sucking and/or dragging the adjacent charge from its blasthole.

In the strong massive granite at the URL, charge dislocation is likely only in a geological fracture zone or if an Inadequate burden distance or blasthole spacing were to be selected. The probability that one charge can blow, suck or drag an adjacent later-firing charge out of its blasthole can be reduced by using a suitable type and amount of stemming.

The initial blast design was based upon 73 blastholes drilled in four concentric rings (see Fig. 5). The inside ring (Ring 1) contained an asymmetrical burn cut incorporating three 89-mm-diameter relief holes (see Fig. 6). The collar burden and toe burden distances are shown in Fig. 7.

Blastholes in Rings 1 and 2 were charged with 32-mm cartridges of FORCITE 75%. Blastholes in Ring 3 were charged with untamped cartridges of 25-mm FORCITE 75%. Each perimeter blasthole contained a 25-mm primer cartridge of FORCITE 75% and two or more cartridges of XACTEX. Table 2 shows blasthole length, relief hole length, charge mass, stemming length and powder factor for initial 2-m-deep rounds. Each blasthole had 300 mm of effective subdrilling, and relief holes were drilled 300 mm deeper than the blastholes (see Fig. 7).



DELAY NUMBER	CHARGE	COLLAR LENGTH
1,2,3	FORCITE 75% x 32mm	200 mm
4,5,6	"	400 mm
7,8,9,10	"	500 mm

Fig. 6 Initial full-face blast design for cut and helper holes

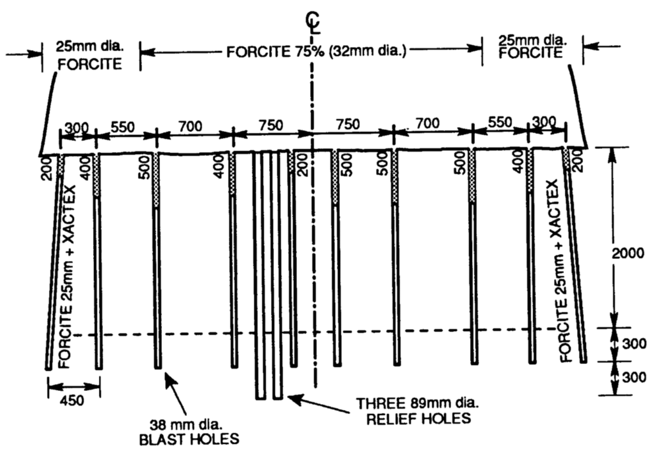


Fig. 7 Section through full-face round—final stemming configuration

Each charge was initiated at its base by a long-period delay MAGNADET detonator having 4-m-long leads. Figs. 5 and 6 show the allocation of delays.

A relief hole becomes less effective if it is allowed to fill with water. This is because the strain wave that propagates outwards from the cut blasthole is far more efficient when it encounters a rock/air rather than a rock/water interface. When it was necessary to expel water from the cut, a small charge (about cartridge) was placed in the bottom of each relief hole containing water and initiated on zero delay.

BLAST MODELLING

A principal attraction of blast modelling is that the effectiveness of an initial blast design and the influence of design modifications can be predicted quickly and before a blast is fired. C-I-L's blast simulator, SABREX, indicated the following:

1. The initial design would break to grade and fragmentation would be satisfactory.
2. The replacement of 700 mm by 200 mm collars in perimeter blastholes would encourage

Table 2 Design blast parameters for 2 m (advance) rounds

Ring	Number and Length of Relief Holes	Number Length of Blastholes	Drilled Metres	Explosive Type	Number of Cartridges per Blasthole and Charge Length		Charge Mass (kg)	Charge per Ring (kg)	stemming Length (m)
	3×2.6		7.8						
1 (+Cut)		13×2.3 m	29.9		11	2.2 m in 3 cut holes	2.29	27.7	0.2 in 3 cut holes
			FORCITE 75%	10	2.0 m in next 3 holes	2.08		0.4 in next 3 holes	
			32×200 mm	10	1.8 m in other 7 holes	2.08		0.5 in other 7 holes	
2		11×2.3 m	25.3		11	1.8 m	2.29	25.2	0.5
				FORCITE 75%					
3		17×2.3 m	39.1	25×200 mm	9	1.8 m	1.22	20.8	0.4
				FORCITE 75%	1				
4		32×2.3 m	73.6	25×200 mm		1.4 m	0.58	18.4	0.7
				plus XACTEX	2				
	TOTAL	7.8 of of 89 mm relief hole+ 167.9 m of 38 mm blasthole					TOTAL	92.1	
			ACTUAL POWDER FACTOR=2.60 kg/m^3 EQUIVALENT POWDER FACTOR=2.55 kg/m^3 (ANFO BASIS)						

inter-blasthole splitting and promote fragmentation in the collar region. (For perimeter collar lengths of both 700 mm and 200 mm, the extent of backcracking was minimal, with only one crack extending backwards for an estimated 200 mm.)

3. The overbreak zone from Ring 3 charges was within that created by perimeter charges.

BLAST MONITORING

Blast monitoring was carried out during the ini tial stages of shaft excavation. Objectives of the monitoring program were

1. to optimize blast design, and

2. to determine ground vibration levels close to blasts.

The vibration monitoring system developed by Australia's Julius Kruttschnitt Mineral Research Centre allowed AECL

1. to determine the time at which each charge actually fired and, hence, Instances of out-of-sequence detonations and "crowding";
2. to detect sympathetic detonations and "misfires";
3. to further improve the blast design by identifying the number of charges that contribute substantially to effective rock breakage; and
4. to measure vibration levels.

(In the present context, a "mistire" is a charge that fails to contribute effectively to rock breakage. Many such misfires are the result of one charge breaking more than its assigned burden and, therefore, of damaging the rock surrounding an adjacent later-firing charge. If a later-firing charge is partially or completely robbed of its burden or is dislocated within Its blasthole, its breakage capability is reduced. The resulting strain energy imparted to the rock mass is thus small and easily identified on the vibration record. The practice of bottom priming virtually ensures initiation of the primer and at least the bottom portion of each charge column.)

Monitoring technique

Instrumentation comprised a series of geophones installed in four 100-mm-long, 63-mm-diameter holes in the shaft wall, some 11-23 m from the blasts. The geophones were connected to a Racal FM tape recorder located on the 240-m level, some 30-40 m from the blasts. The tape was set running about 10 minutes before each blast. After blast-generated fumes had been cleared, the tape recorder was retrieved to enable blast analysis to be carried out on the surface.

Analysis of data

The first step of the analysis was to extract the data from the tape recorder through a Norland waveform analyser to floppy disc for permanent storage. Further analysis was then carried out using the waveform analyser.

The waveform was interrogated at a sampling period rate of 50 μs and each period was assigned to a particular delay number. The maximum impulse amplitude was extracted. The degree of confidence that may be attached to the interpreted detonation events depends on the distribution of the geophones relative to the explosive charges. The complexity and scale of blasting also influences the relative ease of Interpretation. Once the impulses have been assigned, sympathetic detonations, misfires and low-detonation charges can be identified. After analyzing the data, a graphic printout of the complete waveform was made, with a sampling rate of 500 μs, using a Hewlett-Packard X−Y plotter.

The surface mounting of geophones on the face of the shaft complicated the nature of individual vibration impulses. Each detonation event was characterized by an initial arrival followed approximately 2 ms later by a generally large amplitude reflector wave. Surface mounting the transducers precludes accurate strain energy amplitude analysis, as the instruments are subject to free-face reflections rather than pure body wave energy.

As it was not practicable to set a trigger at time zero of a blast, timing data were not absolute. The three zero (nominal 157 ms) delays, used to blow water from the relief holes, were taken as the reference point for timing all detonations.

BLAST PERFORMANCE

Result of blast monitoring

Monitoring five of the early rounds showed the cut configuration worked effectively: each of the three cut charges detonated and contributed to the breakage process. Monitoring also indicated that the perimeter charges performed very well.

The most consistent observed blast malfunction was associated with misfired charges in Ring 2. The evidence indicated that either the burden distance or blasthole spacing was smaller than optimum.

Helper charges (adjacent to the cut—see Fig. 6) were damaged in two blasts, indicating that the design spacing between the cut and the helper charges (300–370 mm) was too small. Three of the five monitored blasts had only two helper blastholes. This straying from design was necessitated by local conditions and also by the need to alternate placement of the cut in successive rounds to eliminate blasthole overlap and bootleg problems. The fact that these three blasts (with the reduced number of helper blast-holes) pulled to full depth also supports the view (resulting from the analysis of vibration data) that helper charges could have broken a burden greater than the design burden.

Although monitoring revealed detonation deficiencies in Rings 2 and 1, these first two rings consistently broke to grade. For this reason and because of the need to limit the number of trial blasts, monitoring results did not justify changes to the blast design.

Precise interpretation of detonation events was made difficult by the scatter of the detonators, especially for the higher delay numbers. The possibility of cumulative scatter preventing progressive relief of burden had been anticipated in the initial design and obviated by using at least alternate delays for dependent charges in adjacent rings (see Fig. 5). As is the case with all pyrotechnic delays, scatter does not allow perimeter charges to create the optimum post-split effect. Electronic delay detonators have the potential to fulfil this requirement. Toward the end of the project, two blasts fired with electronic detonators indicated that very accurate delay times could contribute appreciably to the degree of success of blasting full-face shaft rounds.

Modified blast design

The initial blast design was modified slightly once the results of the first few blasts were observed and as a result of the blast monitoring. The modified design differed from the initial design in the following minor ways:

1. The distance between the cut blastholes and relief holes was increased from 170 to 180 mm.
2. In each Ring 3 blasthole, every third cartridge was tamped. This practice was based upon the observed lengths of perimeter ring bootlegs and was verified by C-I-L's SABREX analysis.
3. Based upon C-I-L's SABREX analysis, stemming lengths were reduced from 500 to 400 mm for Ring 3 blastholes and from 700 to 200 mm for perimeter ring blastholes.
4. In perimeter blastholes, each base charge was increased from one to two cartridges of 25-mm FORCITE 75%. For lookout angles exceeding 4°, the base charge was increased to three cartridges (to cope with the larger toe burden).
5. For 3.5-m rounds, the radius of Ring 1 was increased from 750 to 850 mm and another blasthole was Inserted into this ring; also the 25-mm-diameter primer in each Ring 3 blasthole was replaced by a 32-mm primer.

Important statistics for Blast No. 63, this being one of the better 2.3-m blasts, are presented in Table 3. Blast performance, as determined by breaking to grade, fragmentation, muck-pile looseness and wall quality, was generally good. Several 1.0−, 2.3−, 3.0− and 3.5-m rounds were successfully fired with good advance (3.5 m was the maximum extent of the jumbo feed shells) using the same blasthole pattern and initiation sequence. For this range of drilled lengths, Table 4 shows the percentage advance, the mean powder factor and the % half barrels for all the 89 full-face blasts taken in the shaft extension project.

Whilst six of the ten 3.5-m rounds had to be reblasted, the four successful rounds advanced

Table 3 Important statistics for Blast No. 63

Average blasthole length	2.31 m
Surveyed average advance	2.20 m
Percentage pull	95%
Volume overbreak	6.5%
Percentage of half barrels	49%
Powder factor	2.94 kg/m^3

an average of 96% of the drilled length and gave an average of 32% half barrels. Reblasting was necessary because the 3.5-m rounds necessitated greater drilling precision. Once the crews became experienced at drilling the 3.5-m rounds with the required precision, better results were obtained. Had excavation proceeded deeper, the expertise of the crews would have increased further, probably to the extent that would have enabled 3.5 m rounds to be pulled with consistent efficiency.

Blasts produced little flyrock. The Galloway stage was located as close as 15 m from the face without sustaining damage. This lack of damage is consistent with

1. use of the burn cut rather than the cone cut, and
2. careful selection of stemming lengths and delay sequence.

CONCLUSIONS

In the strong massive granite at the URL, both high production potential and minimal overbreak and flyrock were achieved by

1. developing a good initial blast design, which was subsequently Improved by applying state-of-the-art blast modelling; and
2. implementing the designs with considerable care and precision.

The key to the success of blast designs was a triangular burn cut that had been developed to minimize sympathetic detonation and dynamic pressure desensitization. The high delay numbers of the long-period detonators exhibited appreciable delay time scatter and, therefore, encouraged the use of alternate delays for dependent charges.

Blast modelling allowed the design to be tested before rounds were actually fired in the shaft. Also, design changes could be evaluated on the simulator without risk of delay to the sinking operations.

Blast monitoring during the initial stages of the shaft extension project Indicated out-ofsequence detonations, sympathetic detonations, misfires and high vibrations were not creating problems in the round.

Ten rounds drilled with 3.5-m blastholes required some remedial blasting. The ten

Table 4 Statistics for full-face rounds

BLAST PARAMETERS		DESIGN ADVANCE (m)			
<1.5	1.5 to<2.0	2.0 to<2.5	2.5 to<3.0	>3.0	
No. of blasts in range	15	22	37	2	10
No. of blasts refired	0	1	3	0	6
Mean length drilled (m)	1.36	2.18	2.33	3.01	3.49
Mean surveyed advance (m)	1.20	1.88	2.11	2.71	3.28
Mean surveyed volumes (m^3)	21.01	32.48	37.61	48.39	62.02
Mean explosive mass (kg)	66.70	109.78	113.90	153.23	192.64
Mean powder factor (kg/m^3)	3.16	3.14	3.00	3.17	3.32
Mean % advance	89.8	86.1	90.7	90.0	94.0
Mean % of half barrels	20.2	34.0	36.1	25.9	29.1
Mean overbreak (mm)	123	154	167	167	318

Notes: 1) Reblasts, flattenings and sump blasts not considered in analysis
2) Overbreak includes drill lookout and is expressed on the 2.3-m shaft radius,
3) Three blasts over 3.0 m were reblasted twice.

blasts pulled an average of 3.3 m (94%) and left an average 29% half barrels on the shaft wall.

References

1. Simmons, G.R., 1986. Atomic Energy of Canada Limited's Underground Research Laboratory for nuclear waste management. In: Proceedings of International Symposium on Geothermal Energy Development and Advanced Technology, Toholu University, Sandai, Japan.
2. Katsube, T.J. and Hume, J.P., 1987. Geo-technical studies at Whiteshell Area (RA-3). CANMET Mining Research Laboratories Divisional Report MRL-87–52.
3. Hagan, T.N., 1979. Understanding the burn cut—a key to greater advance rates. In: Proceedings 2nd International Tunnelling Conference, London, UK.
4. Hagen, T.N., 1979. Designing primary blasts for increased slope stability. In: Proceedings 4th International Rock Mechanics Conference, Montreux, Switzerland.

Geotechnical processes for security shafts of the Premetro tunnel under the River Scheldt at Antwerp

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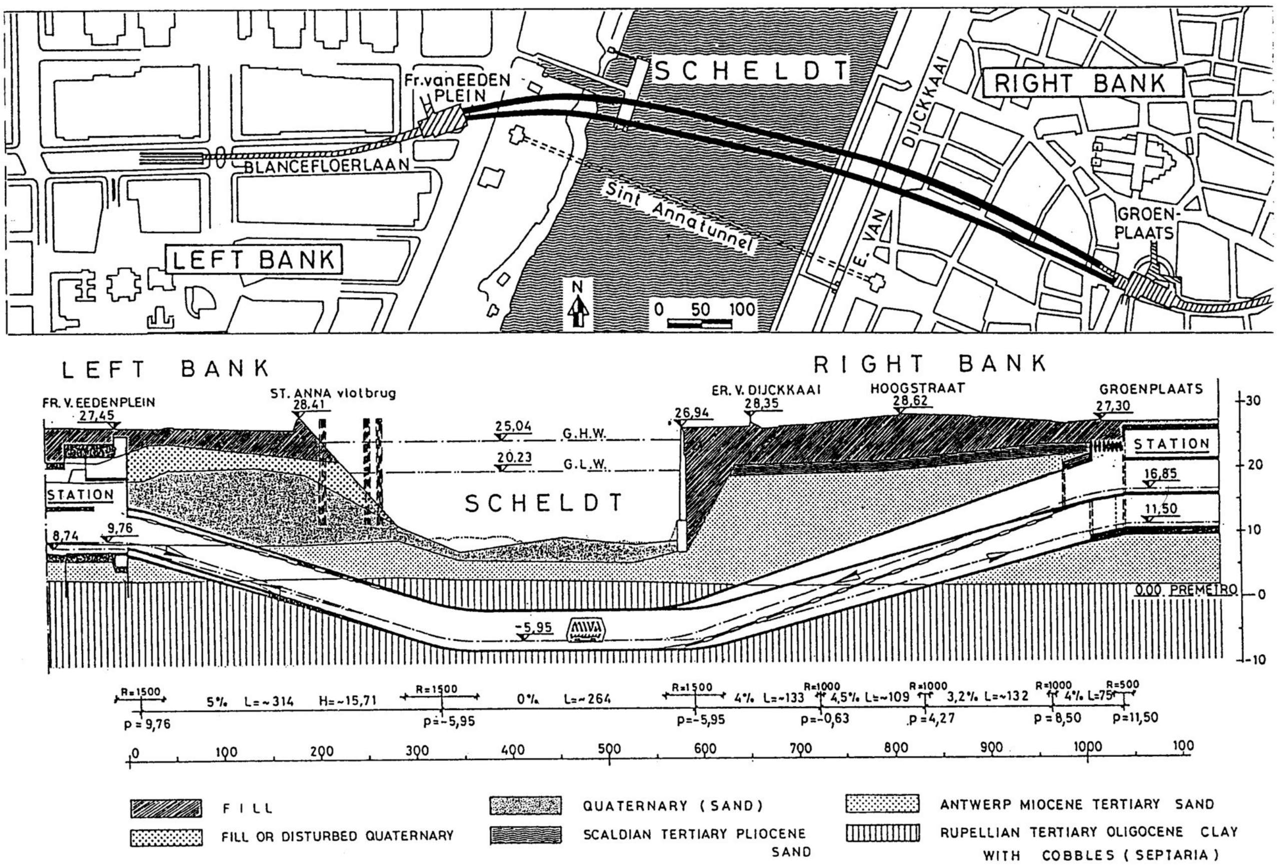

Fig. 1: Location and vertical alignment of the new PREMETRO tunnel/Longitudinal geological section

SUMMARY

The real hydroshield tunnelling for the Antwerp Premetro tunnels under the River Scheldt started in October 1986 and finished April 1st, 1988.

The twin tubes are situated between two stations over a length of about 1,000 m of which 375 m is under the River Scheldt. The maximum depth of the tunnels is 32 m below average high water level.

In the quay wall vicinity, two vertical extraction shafts were constructed to a depth of about 31 m. These shafts were separately connected with the tunnels, to ensure adequate smoke evacuation in case of fire.

Several innovative techniques were applied, such as groundwater lowering, shaft construction with prefabricated tunnel segments, injections and very high pressure grouting were carried out in the fill or disturbed quaternary, and the tertiary sand and the fissured tertiary clay (called 'Boom'—clay). The connection of the shafts with the tunnels was achieved by the thrustjacking method.

The article does not only broach the application of these techniques but also the influence on the quay area, comprising an underground gas pipeline and the flood embankment wall. This influence was checked by means of extenso-meters.

1. INTRODUCTION.

At Antwerp a new PREMETRO tunnel has been constructed in order to link the old town center on the Right Bank of the River Scheldt to the residential area on the Left Bank. This in addition to the existing tunnels (2 road tunnels, a railway tunnel and a pedestrian tunnel).

Traffic studies have shown that the creation of a tramway tunnel between Groenplaats and F. van Eedenplein was the most adequate solution to meet the present needs (figure 1). From an economic point of view, the costs involved in building a tramway tunnel were considerably lower than those involved in other similar alternative solutions.

The new Scheldt tunnel consists of two adjacent tubes each having an internal diameter of 5.70 m. Both tubes were constructed using the shield method. The hydroshield has an outer diameter of 6.80 m. The total length of the single track is 2, 010 m of which 2×375 m is situated under the river bed. The superelevation is a max. 5.5 %. The horizontal tunnel section under the river is about 32 m below mean high water level. The effective protective soil cover above this tunnel section is about 7 m. On the Right Bank the tunnels were situated at about 5 m below the existing quay wall on the Right Bank.

In the quay wall vicinity, two vertical extraction shafts were constructed to a depth of about 31 m. These shafts were separately connected with the case of fire. tunnels, to ensure adequate smoke evacuation in

2. SOIL CONDITIONS.

From an extensive soil investigation with borings and static cone penetration tests it appeared that in the vicinity of the quay wall on the Right Bank the Substrata have the following geological characteristics (fig. 1).

Fill.

Immediately behind the quay wall fill was found down to its foundation level. Further behind the quay wall a top layer of fill and disturbed soil with a thickness of about 7 m was present. The fills were very heterogeneous and consisted of sands, stones and mud.

Quaternary.

In front of the quay wall a mud layer was found on the bottom of the river together with some 3 m of quaternary sands.

Tertiary.

- Scaldian Pliocene Tertiary; under the Right River Bank pliocene glauconitic fossiliferous sand was encountered from a depth of about 7 m below street level down to a depth of about m.
- Antwerp Miocene Tertiary; this dense layer of alluvial sands containing hard shells and shell pieces, was found to a depth of 25 m below street level.
- Rupelian Oligocene Tertiary (Boom type clay); from a depth of about 25 m down to a depth of more than 75 m a stiff fissured overconsolidated clay was found. Directly under the quay wall on the Right Bank the tunnels were completely situated in this clay layer.

3. GENERAL DESCRIPTION OF THE SECURITY SHAFTS.

Considering on the one hand the 1,000 m long Scheldt twin tube tunnel with gradients up to max 5.5 % and on the other hand the use of tramcars and rapid tramcars by which fire-risk is not excluded, the Premetro Study Office has carefully investigated the problem of the safety and passenger evacuation.

Bearing this in mind the tunnel is coated with a fire resisting and acoustical layer based on rock-wool fibres. The layer has a minimum thickness of 20 mm and a fire resistance of Rf=2h.

On the other hand, some special requirements are stipulated for the cables of the electro-mechanical equipment in order to restrict flame extension. Moreover, two vertical security shafts are constructed at a distance of about 1/3rd of the tunnel length, reckoned from the Groenplaats Station (see figure 2).

These 31 m deep. vertical shafts with both an inner diameter of 2 m are connected separately with each Scheldt tunnel and run at street level into a common ventilation room.

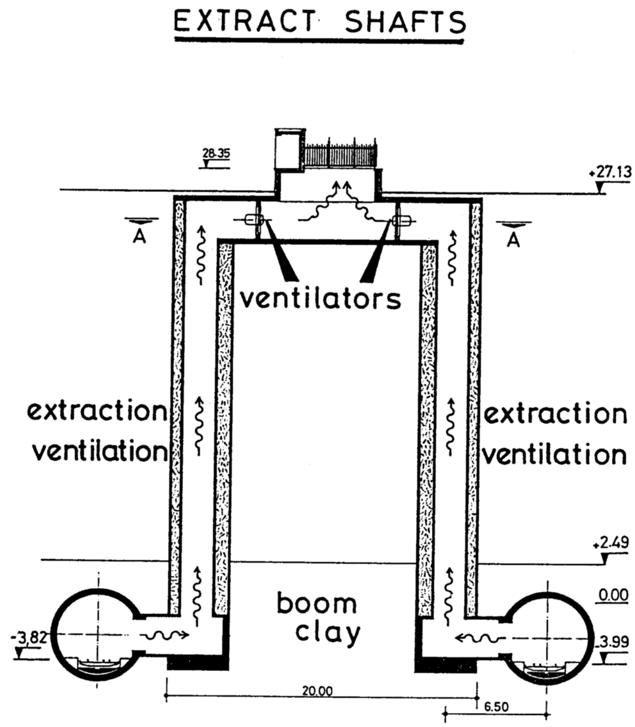

Fig. 2: Cross section of the extract shafts

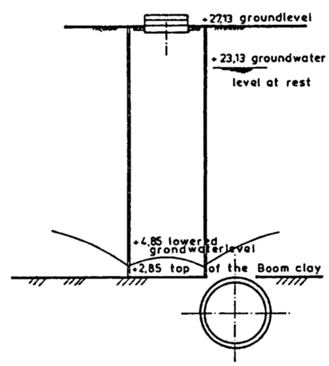

fig.3.

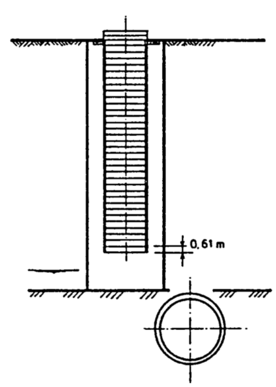

fig.4.

An extraction ventilator with an output of 200,000 m^3/h is installed for each tunnel. These extraction ventilators together with the extractor ventilator in the stations vicinity, will admit to force up the existing natural ventilation in the Scheldt tunnels; this will facilitate the passenger evacuation through a smoke proof emergency exit in case of fire. Moreover the security shafts will allow a prompt intervention of the fire brigade.

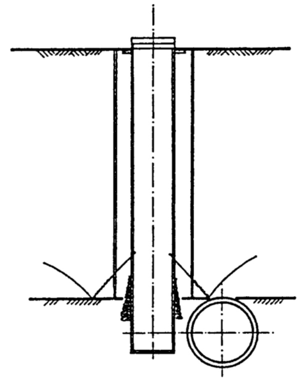

fig.5.

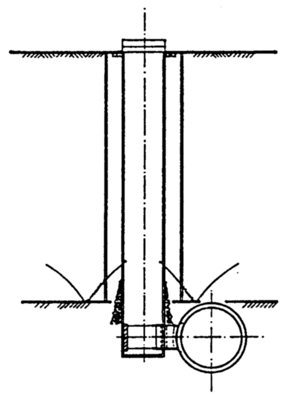

fig.6.

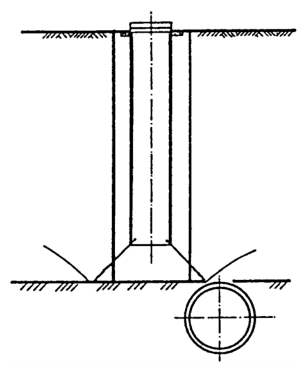

fig.7.

4.
EXECUTION METHOD.

An execution method with several innovative techniques has been proposed by the n.v. Smet-Boring for the installation of the extraction shafts. This method can be summarized as follows :

- General groundwater lowering to about 2 m above the top of the Boom clay layer (fig. 3);
- Mechanical excavation in passes of 0.61 m and installation of a temporary lining with segments of precast concrete, till 1. 50 m above the lowered groundwater level (fig. 4);
- Installation of an additional groundwater lowering system (fig. 5) and of vertical groutcolumns sealing the transition zone between the Thertiary sands and the Boom clay (fig. 6);
- Further mechanical excavation in passes of 0.61 m and installation of a temporary lining with segments of precast concrete, till a depth of about 31 m (fig. 7);
- Horizontal boring following the pipe jacking method (fig. 8);
- Installation of a permanent lining of reinforced concrete provided with a continuous steel plate (fig. 9);

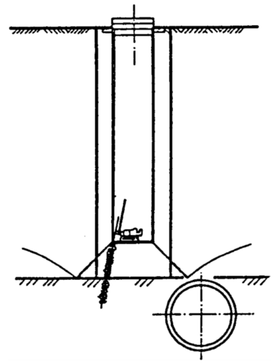

fig.8.

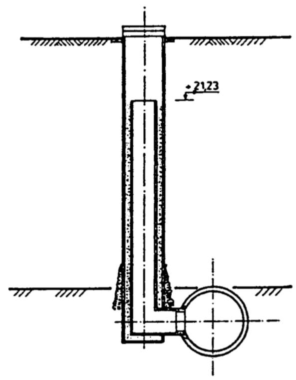

fig.9.

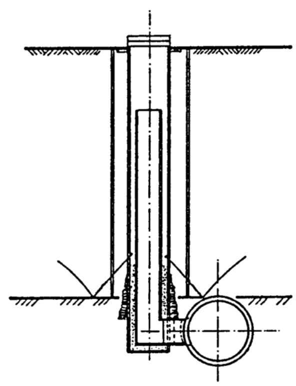

fig.10.

- Opening of the tunnel wall and installation of a ring beam of reinforced concrete (fig. 10);
- Excavation with “Berliner Bauweise” of the ventilation room.

5.
GROUNDWATER LOWERING.

Before any excavation was started. 6 discharge wells were installed around each shaft. They consisted of a P.V.C.-tube with an inner diameter of 160 mm and a filter length of 1 m situated just above the top of the Boom clay layer. The bore-holes with a diameter of 0.40 m were drilled by the direct flush method.

According to the calculations a groundwater lowering to about 2 m above the top of the Boom clay layer was expected. However shortly after starting the groundwater lowering system it seemed that the expected water level could not be obtained and 11 additional discharge wells had to be installed in order to obtain the projected water level.

After the shaft was lowered to 3.5 m above the top of the Boom clay layer, 24 additional filter elements were installed from the bottom of each shaft. They consisted of a P.V.C.-tube with an outer diameter of 50 mm and a filter length of 0.50 m,

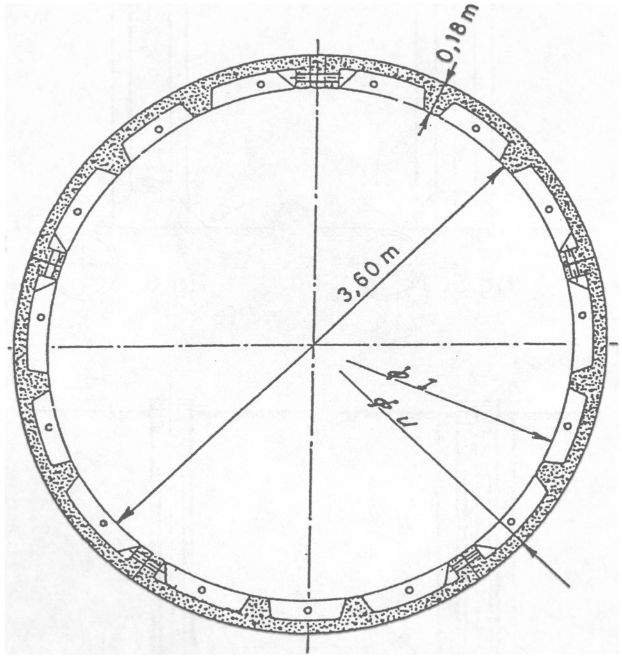

Fig. 11: Typical section through the temporary lining of the shaft

situated just above the top of the Boom clay layer. The boreholes were installed manually under an inclination of 40° with the vertical. All filter elements were connected to a ring pipe. Within this pipe a negative pressure of about 7 m water height was installed by means of vacuum pumps.

With the help of this additional filter elements, the groundwater level could be lowered to less than 0.50 m above the top of the Boom clay layer.

6.
EXCAVATION AND TEMPORARY LINING.

The excavation of the soil was carried out in passes of 0.61 m by means of an hydraulic excavator provided with a vertical telescopic arm. The walls were trimmed manually in order to limit the thickness of the zone between the soil and the temporary lining to a few centimeters.

Each temporary lining ring consists of 6 segments and a key, which were connected to each other by means of bolts (fig. 11). The lining has an inner diameter of 3.60 m and an outer diameter of 3.96 m. These dimensions have been chosen as a function of the horizontal boring which had to be performed from the bottom of the shaft.

The elements were hung on the last installed ring by means of vertical bolts. The individual segments were also bolted together in a horizontal direction. Before starting the excavation, a reinforced concrete slab has been installed around each shaft. This slab was necessary to fix the first lining elements during their installation ands before any injection could be performed.

The zone between the soil and the temporary lining was injected with cementgrout at regular intervals. This was necessary to transmit the weight of the temporary lining to the soil by means of friction and to avoid that a too large weight had to be transferred by the vertical bolts.

At the location of the second shaft a six metres thick layer of hardcore was encountered. It was necessary to inject this layer with cement-grout before the excavation could be started.

7.
VERTICAL GROUT COLUMNS.

In order to create a barrier for the remaining water just above the top of the Boom clay layer, a series of grout columns were installed by means of Very High Pressure grouting (fig. 12).

In the first stage a series of rods were introduced down to the requested depth by means of direct flush. The lower end of the rods were equipped with a drilling head and a special ejector valve carrying two nozzles orthogonal to the rod axis. In the

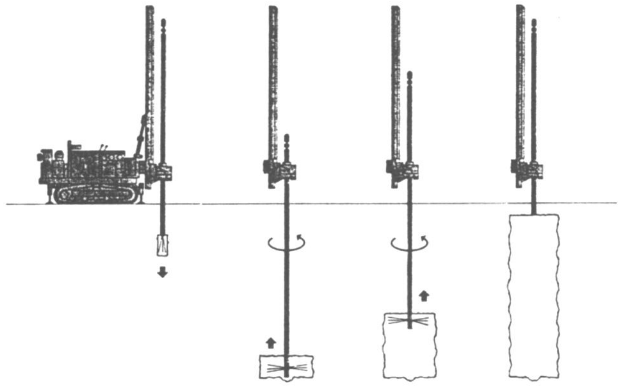

Fig. 12 Installation method of consolidated soil columns by V.H.P.-grouting

second phase the rods are extracted with simultaneous injection. The grout is injected through the nozzles under a pressure of about 40 MPa. The material in place is intensively mixed with the grout to form a mortar. By varying the rotation velocity and the vertical extraction speed it is possible to obtain volumes of treated material of the desired shape and dimension. After hardening of the grout a solid column is obtained.

The grout columns were installed from about 2 m above the top of the Boom clay layer to 2.5 m underneath this layer (fig. 8). Around each shaf t 28 columns were installed. The columns are secant.

As there was no experience available with Very High Pressure grouting in stiff layers an extensive preliminary test program was performed in an old clay pit where the clay was excavated for the fabrication of brick stones. From the results of these tests it could be concluded that the proposed method was appropriate for the creation of a sealing within the transition zone between the Tertiary sands and the Boom clay.

8. HORIZONTAL BORING.

After reaching the bottom level of the shaft a reinforced concrete slab with a thickness of 0.50 m was installed in order to create a temporary plug and to facilitate the installation of the pipe jacking equipment.

Then the temporary lining of the shaft was strengthened over a height of about 5 m by means of shotcrete. Additional vertical reinforcement bars were also placed within the shotcrete layer. This strengthening was necessary to limit the deformation of the shaft lining when a part of it was broken off, for the start of the horizontal boring.

The horizontal boring was performed with tubes of reinforced concrete having an outer diameter of 2.96 m and an inner diameter of 2.50 m and provided with a steel core and a double sealing in the joints. These tubes were pushed into the soil with simultaneous excavation inside. The first tube to obtain a close contact between the horizontal was provided with a special cutting shoe in order boring and the tunnel.

By providing elastic joints between the two tubes of the horizontal boring, slight differential settlements can be accepted between the shaft and the tunnel.

After reaching the tunnel a temporary plug was placed between the cutting shoe and the tunnel. Some reinforcing bars were drilled in the tunnel elements and fixed to the cutting shoe. Afterwards a shotcrete layer was placed over these bars.

9. PERMANENT LINING AND OPENING OF THE TUNNEL WALL.

After finishing the horizontal boring a prefabricated T-shaped element was lowered into the bottom of the shaft and fixed to the tubes of the horizontal boring. These connected the horizontal boring with the permanent vertical shaft. The permanent vertical shaft has an inner diameter of 2 m and an outer diameter of 2.30 m. All the elements are of reinforced concrete and are provided with a steel core. These are welded to each other on the site in order to obtain perfect watertightness. The area between the temporary and permanent lining was filled with a sand-cement mixture.

When the permanent lining was installed, the tunnel wall was opened and a ring beam of reinforced concrete installed. During this operation the tunnel was strutted with a special construction containing flat jacks in order to resist any movement of the tunnel wall.

10.
MEASUREMENTS.

During the entire construction of the extract shaft measurements have been performed by means of vertical extensometers installed at different levels. The maximal observed settlement amounted to 7 mm.

When the first shaft was excavated a few meters into the Boom clay the vertical movements of the bottom shaft were measured during a week-end, in order to check the swelling of the clay. Over a period of almost 3 days a maximum uplift of 3. 2 mm was observed.

11.
CONCLUSIONS.

In rather difficult soil conditions two extraction shafts have been constructed to a depth of about 31 metres.

By introducing several innovative techniques, such as extreme groundwater lowering, shaft construction with prefabricated tunnel segments, injections and Very High Pressure grouting, a safe and economic solution has been obtained for the construction of these shafts.

Measurements performed throughout the construction period of the shafts indicated that rather small settlements took place.

Groundwater response to shaft excavations in decomposed granite

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SYNOPSIS

Of the many shafts that were sunk for the construction of the Hong Kong Mass Transit Railway Island line, nine were excavated up to 30 m deep in granite which was so deeply weathered as to be classed as soft ground down to final formation level and, in some cases, below the toe of the perimeter diaphragm walls. These excavations were made up to 28 m below sea level and within metres of high rise buildings sensitive to dewatering settlement. No dewatering was allowed next to these shafts, so ground water pressures were a major factor in the design of temporary support.

Chosen diaphragm wall panels were instrumented with earth pressure cells twinned with pneumatic piezometers both inside and outside the excavation wall. This paper analyses the data obtained from the pneumatic piezometers as excavation proceeded. The data obtained challenge fundamental assumptions made for design and indicate that considerable savings could have been achieved in temporary support using an observational method of design.

Conclusions are also made on the effectiveness of grout curtains below the walls and on the value of early dewatering ahead of excavation.

INTRODUCTION

The Sheung Wan to Chai Wan Island Line for the Mass Transit Railway (Fig. 1) is the third phase of Hong Kong's predominantly underground urban mass transit system. To limit disruption to the already congested road network on Hong Kong Island, it was decided to build the platforms in 8 m diameter bored tunnels.

These tunnels could often not be accommodated side by side under the roads in this intensely developed urban environment and sites for stations on both sides of the road were not always available (Fig. 2). In many cases, the lower tunnel is up to 30 m below ground level. Consequently, the temporary shafts for the initial tunnel drives and the permanent passenger access off-line station boxes had to be excavated down to a depth of 30m.

The method generally used for construction of the stations was to form a box using permanent diaphragm walls, inside which the floor slabs were cast as excavation progressed in a "top down" sequence. The diaphragm walls were excavated down to a level where a Standard Penetration Test (SPT) "N" value of 200 blows/0.3 m was achieved, for which the local Building Authority allowed a design bearing capacity of 1.2 MPa. In many cases this was 40 m to 50 m below ground level, although one panel was excavated to 56 m depth and is believed to be the deepest structural panel ever to have been concreted.

The station boxes formed the foundations and basements of large high-rise building developments and these were generally surrounded by other buildings in urban areas with ground level not much above sea level, in a city where land prices have often set world records. The perimeter walls were therefore sometimes less than one meter away from existing building foundations sensitive to dewatering settlement. All possible precautions were taken to minimize dewatering, and the walls had to be designed for full hydrostatic water pressures.

Before the lowest floor slab could be cast at a final excavated depth of 30 m there was an 8 m vertical wall span below the penultimate floor slab.

In soft ground this gave rise to quite formidable theoretical and practical temporary support problems.

GEOLOGICAL BACKGROUND

The medium grained granitic rocks of the north coast of Hong Kong Island and the Kowloon Peninsula are described in detail by Strange and Shaw[1]. These granites are known to be intruded in places by basaltic dykes (dolerites), rhyolitic dykes (micro-granites) and quartz veins. Aplites and pegmatites have also been found.

Chemical weathering can alter all these rocks to a state where most engineers (and some geologists) refer to the material as "soil". Where this alteration is limited to kaolinisation of the feldspars, and the original rock-like fabric is still visually

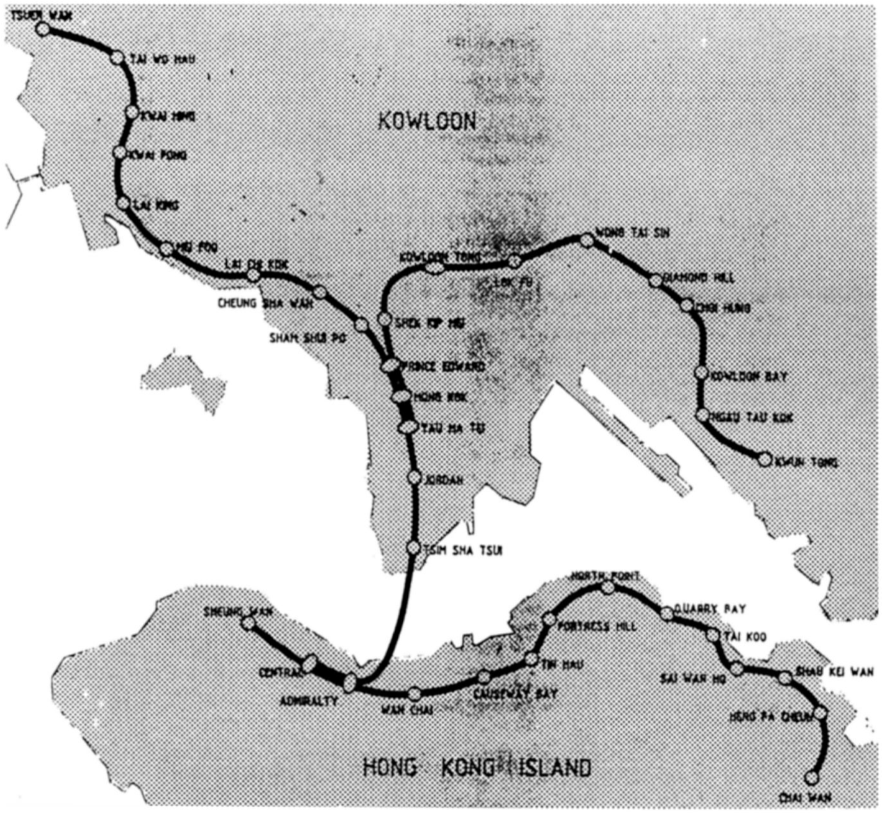

Fig. 1 Hong Kong Mass Transit System

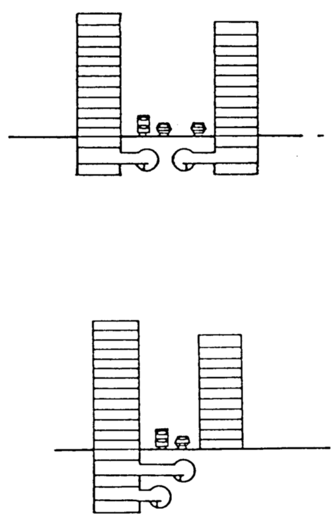

Fig. 2 Tunnel and Concourse Configurations

apparent, the resulting material is called a “saprolite”.[2] The local name is “decomposed granite”.[3] Howat[4] points out that the use of the term “soil” is misleading and should be avoided. He prefers to consider the material as extremely weak rock.

Moye[5] differentiates between a saprolite which disintegrates when immersed in water (completely weathered granite) and one that does not (highly weathered granite). The saprolites studied in this paper were all below the water table and saturated, therefore such a distinction is not strictly applicable. Hencher and Martin[6] introduce the concept of strength differences between completely and highly weathered granite which were specifically excluded in the Moye classification. In order to avoid such ambiguities neither the Moye nor Hencher and Martin terminologies are used in the paper.

Above the saprolites are found soils which can be residual, colluvial (transported), marine or fill. These did not extend below 20 m depth in these case studies and are not considered in this paper.

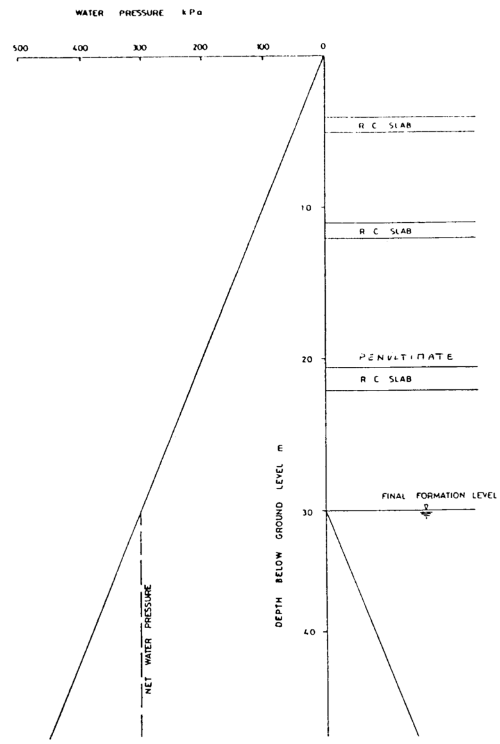

Fig. 3 Water Pressures on Wall Theoretical Model

In the study areas, the ground level rarely exceeds 4 m above Chart Datum, with a maximum tidal range of about 3 m. The water table is very close to ground level, sometimes controlled by fresh groundwater from the hills above, sometimes controlled by the tides.

ENGINEERING PROPERTIES OF DECOMPOSED GRANITE

Decomposed granite has traditionally been considered to lose its cohesion on saturation due to loss of capillary pore water tensions.[7] Howat[4] argued that this conclusion was erroneous and was partly the result of unrealistically severe laboratory saturation procedures, and partly the result of problems of interpretation of test results.

Howat[8] found that the in situ unconfined strength of saturated decomposed granite could vary with void ratio from insignificant values to more than 500 kPa. This strength generally, but not always, increases with depth.[4]

Permeability as measured in the laboratory generally varies from 1× 10–5 to 10–9 m/s depending on texture and void ratio.[3] Values outside these limits were found in situ in the study areas, and these are considered to reflect the presence of discontinuities and intrusions.

THEORETICAL MODEL

Fig. 3 is a sketch of typical 30m deep excavation in the final stage of excavation. The water pressures are assumed to be hydrostatic from ground level outside the excavation since all efforts are made to build a waterproof wall and prevent dewatering effects outside the excavation.

The water pressures are also assumed to be hydrostatic from formation level inside the excavation, since the dewatering system consists of positive pumping from inside dewatering wells extending down to diaphragm wall toe level. The water pressure imbalance is assumed to be maintained by a grout curtain which extends down to bedrock, or well below toe level if rockhead is very deep.

The theoretical net water pressure exerts extremely large moments about the penultimate permanent reinforced concrete slab which could not be resisted by the strength of the wall. In the case histories analyzed in this paper up to three levels of temporary steel struts were used, each level being pre-loaded to 1MN/m run of wall and having a design working load of up to 2MN/m.

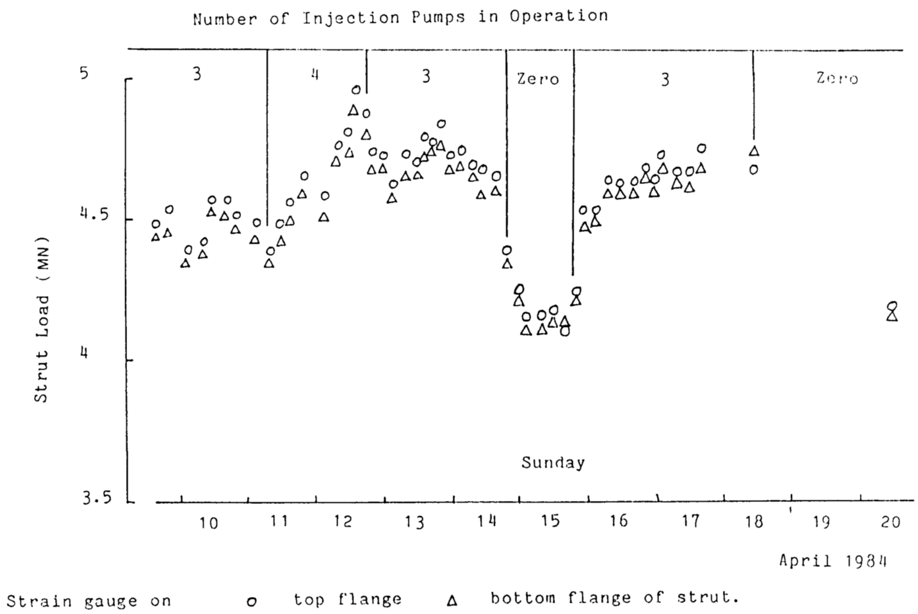

Fig. 4 Increase in Strut Load Due to Ground Treatment Injections—from Howat[8]

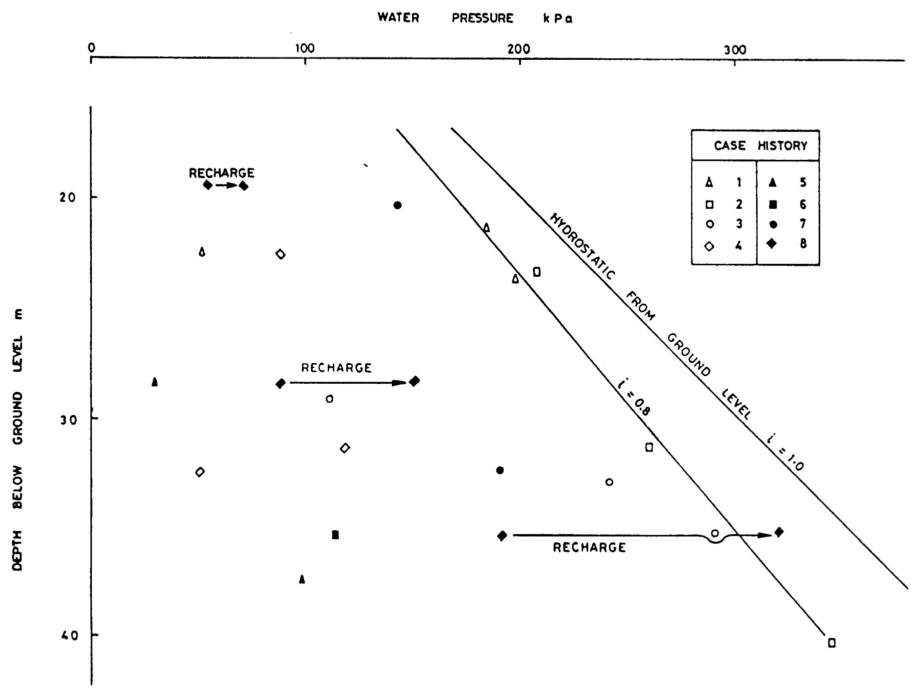

Fig. 5 Water Pressures Recorded on Outside Face of Wall Excavation at Final Formation Level

STRUT LOAD MONITORING

The strut loads were monitored using vibrating wire strain gauges, but the results proved inconclusive. Most strut loads did not increase significantly from the preload values, and there was often a recorded dramatic decrease in load. Because the data from individual strain gauges sometimes indicated negative values, little confidence was placed in the data at the time.

In only one case did the load increase significantly beyond preload value. This was caused by a ground treatment operation which was being carried out on the outer face of the wall. This increase in load was controlled by reducing the number of injection pumps in operation at any one time. Fig. 4 shows the recorded strut load. It is clear that the gauges are responding well at this location.

WATER PRESSURE DATA

Water pressures were measured throughout dewatering and excavation by means of pneumatic piezometers cast into the diahpragm walls as part of Gloetzl earth pressure cell units. These units were positioned on both inside and outside faces of the walls and jacked out from the steel reinforcing cages before concrete was tremied into the slurry trenches.

Fig. 5 shows the actual water pressures measured on the outside face of the diaphragm walls with excavations at final formation level. It is clear that, in many cases, there was a very significant leakage through irregularities in the walls. Standpipe piezometers installed in the saprolites only a few metres distant from the diaphragm walls sometimes picked up drawdown due to this leakage, but it was only a fraction of the values measured at the wall. The instruments in the surficial soils above the saprolites were never affected.

In one case this drawdown settlement led to an unacceptable degree of angular distortion of an old building adjacent to the excavation and a recharge well system was installed to control further differential movement. Fig. 5 shows the effect of this recharge. The restitution of water pressure was almost complete at 35 to 36 m depth, but incomplete above this level.

Howat & Cater[9] found that the design assumption for hydrostatic water profiles inside two advance contract shaft excavations (case histories 1 and 2) was not correct. Despite heavy pumping from deep wells inside the passive block, artesian water pressures were found to have developed, reducing vertical effective stresses to zero or even negative values. Analysis of the pneumatic piezometer data from the inside wall faces of all the other excavations showed that this was a general phenomenon.

The hydraulic gradients inside the excavations have been calculated from the pneumatic piezometer data using the expression,

$$i = \frac{(u_w)_B - (u_w)_A}{z_B - z_A}$$

where u_w =pressure in piezometer A, B in metres head of water
z =depth of piezometers A, B in metres

and the depth to the phreatic surface inside the excavation, using the expression,

$$z_{ps} = z_B - \frac{(u_w)_B}{i}$$

Fig. 6 shows the hydraulic gradient plotted against phreatic surface for the eight case histories.

Fig. 7 shows the net water pressure Δu_w inside and outside the diaphragm walls as the excavation approached final formation level. This has been normalized by dividing by the net water pressure at the same level as the phreatic surface inside the wall. It appears that there is a linear reduction to zero (balanced water pressures) at the diaphragm wall toe.

DEWATERING BEFORE EXCAVATION

Fig. 8 shows the effect of dewatering inside a diaphragm wall box, undertaken before excavation to allow workers to complete piles by hand inside temporary steel liners. A survey of the adjacent tunnel had independently shown a 40 mm lateral deflection, which met with some incredulity at first.

In another case study, dewatering in advance of excavation occurred when a deep inspection shaft was excavated in order to carry out a check on the wall integrity. Fig. 9 shows the effective earth pressure measured in Gloetzl earth pressure cells next to the pneumatic piezometers. It can be seen that, as the active earth pressure on the outside of the wall decreased, so did the "passive" earth pressures on the inside.

One zone of the diaphragm wall in this case study was not affected by this premature dewatering, being protected by a subvertical fine grained saprolitic intrusion. In this case the wall took up a severe deflected shape as the excavation approached final formation level, with an angular distortion of about 1 in 50 (see Fig. 10), and displayed some horizontal cracking. It should be added that, in this part of the excavation, the temporary strutting was not preloaded.

INTERPRETATION

Fig. 5 shows that when the excavations had reached final formation level the hydraulic gradient outside the excavation reached a maximum of 0.8, not the hydrostatic profile (i=1) traditionally used in design. In some cases water pressures far lower than hydrostatic were found, and were attributed to leakages through irregularities found in the diaphragm wall after excavation. This drawdown was found to be limited to the saprolites and was negligible in the surficial deposits above. Even

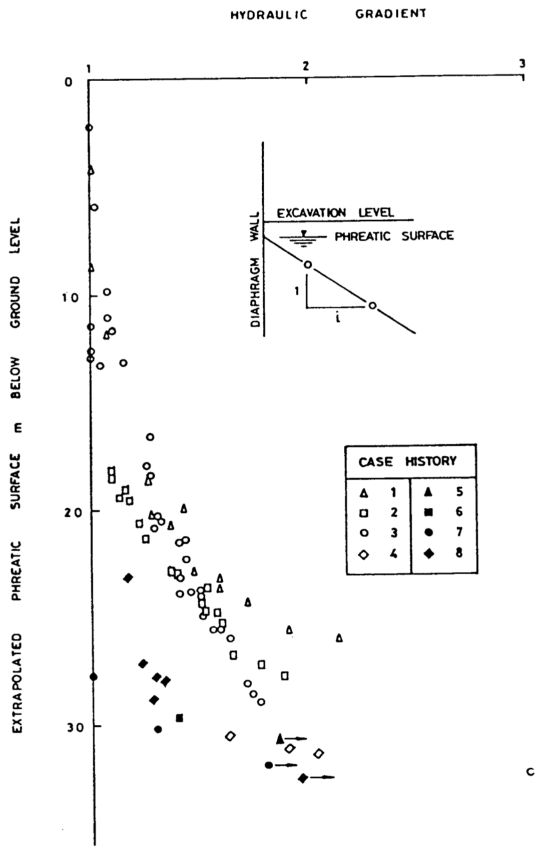

Fig. 6 Hydraulic Gradient Inside Excavation

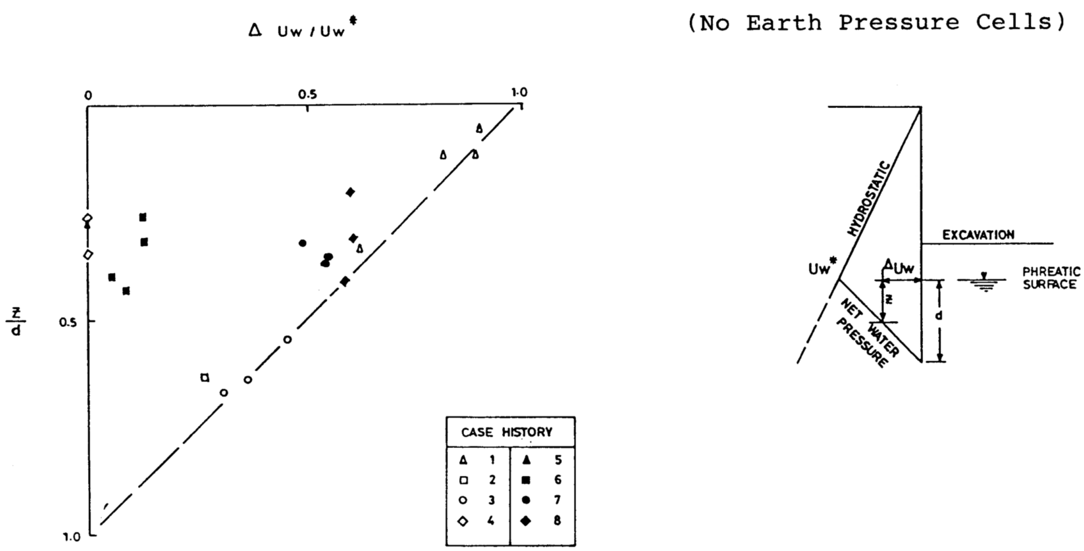

Fig. 7 Net Water Pressure on Outer Face of Wall Below Excavation Level

within the saprolites, the radius of action was found to be small, and in only a few cases did unacceptable settlement occur to adjacent buildings.

Fig. 6 shows that hydrostatic conditions inside the excavation broke down as the phreatic surface approached final formation level, in some cases well before. This occurred whether or not the toe of the wall was chiselled and grouted into underlying bedrock. The high water pressures are therefore considered to be caused by a substantial recharge from permeable discontinuities within the rock mass below.

It could be argued that these high water pressures are experimental errors in the monitoring system since water pressures exceeded overburden pressures and yet "quicksand" conditions did not occur, except in small areas which did not affect the

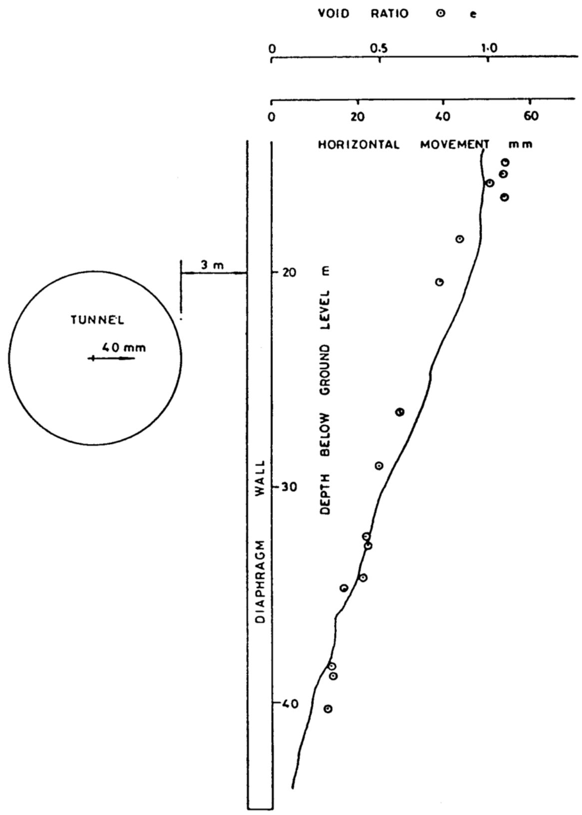

Fig. 8. Wall Movement on Dewatering Case History 9

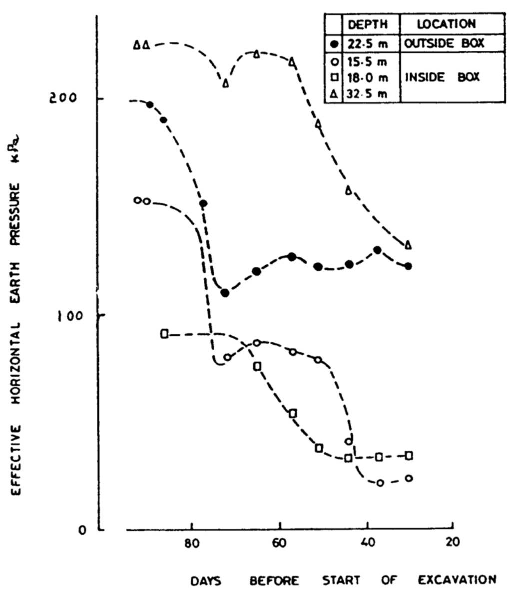

Fig. 9 Reduction Earth Pressure on Both Sides of the Diaphragm Wall on Dewatering—Case History 4

overall stability of the excavations. However, data from the same shaft excavations show that decomposed granite can have high unconfined compressive strength especially at low void ratio 8 which prevents the tendency to liquefy at zero or negative effective stress levels.

Fig. 7 shows that water pressures were balanced at diaphragm wall toe level. It appears that where a grout curtain extended below the diaphragm wall toe, the curtain had no effect.

When compared with the theoretical water pressure distribution shown in Fig. 3, it is clear that the actual water pressure distribution shown on Fig. 7 exerts a far smaller bending moment on the wall. This goes some way to explaining why the monitoring of strut loads indicated either no increase in load after installation and preloading or, in many cases, a dramatic

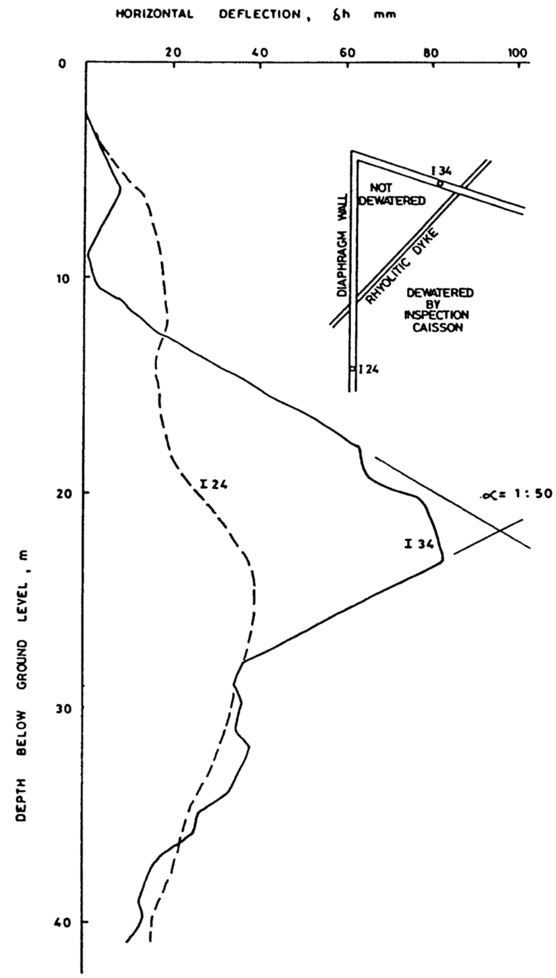

Fig. 10 Wall Movement on Excavation Case History 4

decrease in load. It is not inconceivable that the negative loads indicated by some data were in fact tension induced in an upper layer of struts by preloading of the layer below.

Fig. 8 shows that dewatering prior to excavation can cause relatively large deflections in the diaphragm wall. It was initially thought that this had taken place because the water pressure balance had forced the wall to mobilize passive frictional pressure against the earth inside the box.

Fig. 9 shows that this is not the case. It is more likely that the decomposed granite inside the box was shrinking because of the dewatering, so that the earth pressures were being relieved on both sides of the wall. This shrinkage may be proportional to the void ratio of the decomposed granite.

Comparing the deflected shapes shown in Fig. 10, it seems that advance dewatering is beneficial.

CONCLUSIONS

Experience from the nine excavations in decomposed granite described in this paper has revealed significant deviations from the assumptions of hydrostatic water profiles made in the design of temporary support for these shafts.

The number of sites studied was limited, thus it would be unwise to suggest that these findings apply universally to all deep excavations, even in the same geological conditions. The case histories studied do indicate, however, the value of instrumentation as a means of checking the validity of basic design assumptions, and suggest that the stresses and moments acting on the walls were far lower than anticipated.

It may well be desirable to plan for less temporary support in future excavations in similar geology, equip the walls with more extensive instrumentation than described in this paper, and verify the validity of the new assumptions as excavation progresses. The potential savings in time, money and particularly construction convenience are considered to outweigh the capital expenditure and monitoring costs involved.

Early dewatering within the box before excavation caused the walls to deflect. This appears to be caused by shrinkage of the ground within the box structure, not by mobilization of passive pressures, so it is, to a large extent, inevitable. The magnitude of the wall deflection depends on the void ratio of the saprolite within the box.

Early dewatering is advantageous, since it produces a smoother deflected wall profile and may prevent overstressing and cracking of the partially strutted wall at later stages of excavation. The dewatering system must be extensive enough, however, to be effective throughout the whole of the inside of the box, which may be compartmentalized by vertical geological aquicludes.

The efficacy of many grout curtains extended below the toe of the diaphragm walls is seriously questioned. Ducts installed in the diaphragm wall panels were useful, however, for checking wall integrity by sonic surveying techniques. These ducts can also allow ground treatment to be made in exceptional circumstances.

Acknowledgements

This study would not have been possible without the considerable efforts of Mr. Frederick Wan and the technicians of the Geotechnical Unit, Materials and Quality Control Section of the Hong Kong Mass Transit Railway Corporation, and the patience and forbearance of all the Resident Engineering Staff.

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References

1. Strange P.J. and Shaw R.Geology of Hong Kong Island and Kowloon, Hong Kong Geological Survey Memoir No. 2, Government of Hong Kong, 1986. p. 134
2. Nogami J.S. Peculiarities of Geotechnical Behaviour of Tropical, Lateritic and Saprolitic Soils. Progress Report (1982–1985) of the Committee on Tropical Soils of the ISSMFE, Brazilian Society for Soil Mechanics, Sao Paulo, 1985. p. 3–8.
3. Lumb P. The properties of decomposed granite. Geotechnique, vol. 12, no. 3, 1962. p. 226–243.
4. Howat M.D. Completely weathered granite—soil or rock? Quarterly Journal of Engineering Geology, vol. 18, no. 3, 1985. p. 199–206.
5. Moye D.G. Engineering geology for the Snowy Mountains Scheme. Journal of the Institution of Engineers, Australia, vol. 27, 1955. p. 287–298.
6. Hencher S.R. and Martin R.P. The description and classification of weathered rocks in Hong Kong for engineering purposes. Proceedings of the Seventh Southeast Asian Geotechnical Conference, Hong Kong, vol. 1, 1982. p. 125–142
7. Baynes F.J. and Dearman W.R. The relationship between the microfabric and the engineering properties of weathered granite. Bulletin of the International Association of Engineering Geology, no. 18, 1978. p. 191–197.
8. Howat M.D. The in situ strength of saturated decomposed granite. Proceedings of the 2nd International Conference on Geomechanics in Tropical Soils, Singapore, vol. 1, 1988. p. 311–316.
9. Howat M.D. and Cater R.W. Passive pressure in completely weathered granite. Proceedings of the First International Conference on Tropical, Lateritic and Saprolitic Soils, Brasilia, vol. 2, 1985. p. 371–379.

Combined grouting and depressurizing for water control during shaft sinking

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SUMMARY

When shaft sinking at depths of several hundred metres groundwater inflow problems can arise from zones of relatively low permeability e.g. 10^{-6} m/sec. The options available for ground treatment are conventionally dewatering, grouting with cement and/or chemical grouts or freezing. The choice is made after careful consideration of a number of factors such as the nature of permeability, stability of the ground, likely level of water inflow, the environmental situation and cost.

Whilst ground freezing will produce "dry" conditions for sinking, neither dewatering nor grouting can ever be fully effective and hence the shaft will have to be sunk with some degree of residual inflow. Grouting, for example, has to be carried out to a very high degree of efficiency to provide the necessary reductions in inflow for acceptable sinking.

From theoretical calculations it is shown in the paper that a combination of grouting and depressurising (rather than dewatering) the formation can be very effective, since the weaknesses of each system used separately are counteracted by the strengths of the other system when used jointly.

Several case histories are reviewed where such a combined treatment was employed, either using deep wells drilled from surface (as in New Mexico, USA) or with wells drilled as a "cover" from the shaft sump (as at Selby Mine, UK).

INTRODUCTION

When sinking deep shafts through water bearing strata, which may also be potentially unstable, there are usually a number of options for ground treatment to eliminate, or at least minimise groundwater inflow and allow shaft sinking to proceed in tolerable conditions. Where the inflows are greater than can be coped with by simple sump pumping the methods which have traditionally had the most widespread use are grouting and ground freezing. Deep well dewatering/depressurising, which is the subject of this paper, has been used on very few occasions. The advent of drilled shafts does present another option and when used in conjunction with a mudfilled hole eliminates the need for additional ground treatment in the aquifer zones. However, to date, this technology has not been applied in the United Kingdom.

Depending on the particular hydrogeological conditions which exist, and these will have to be determined by a thorough site investigation, or be known through previous workings in the same area, the choice between grouting and freezing may be more or less pre-determined. For example, where extensive alluvial deposits as in China and Germany, or a very thick and continuous rock aquifer (e.g. Bunter Sandstone at Selby Mine) exist from surface level then from both technical and economic points of view ground freezing will be the preferred method.

If a single or series of thinner, but still heavily waterbearing zones are met at greater depths then ground freezing would be difficult, inefficient and very expensive, and grouting would offer the best approach. However, in such circumstances the permeability of the aquifer zones may be primarily due to intergranular pores and very fine fissures, which can only be sealed with a highly penetrating chemical grout. In these cases grouting can become very time consuming in order to achieve the level of permeability reduction that is considered acceptable.

It is in such instances that the incorporation of depressurising wells might be considered. If used on their own they can prove effective in reducing the initial aquifer pressure and hence the potential inflow to the shaft but once the shaft excavation is within the aquifer the effects of drawdown at the shaft and at the wells interfere and the effectiveness of the well system can be considerably reduced. If however the well system is used in combination with grouting, the merits of the two systems reinforce one another and their respective deficiencies when used alone are minimised. For example the degree of grouting required is not so demanding so the time involved is significantly reduced.

The use of well depressurising systems can involve either deep wells drilled from the surface which contain submersible pumps, or shallow wells drilled in a 'cover' pattern from within the shaft where the aquifer pressure itself is used to develop the flow. Instances where such systems might be used are:

1. where very heavy inflows are anticipated from discrete aquifer zones and there is a need to make grouting efficiency less onerous.
2. where weakly consolidated strata is encountered at depths and could collapse under the hydrostatic pressure even after grout treatment.
3. where conventional hydrostatic concrete shaft linings could be employed at greater depths i.e. say≥600 m, if lower backwall grouting pressures could be employed.

This paper examines the use of depressuring wells from the theoretical standpoint and in particular when used in combination with grouting. Several case histories are then discussed which cover the three main applications mentioned above.

THEORETICAL CONSIDERATIONS

Effectiveness of Grouting

Before looking at the effect of combined grouting and de-pressurising, it is worth examining each of them as used separately. It should be remembered that the estimates of water inflow are made on the basis of the formation being homogeneous—a situation never achieved in practice. None the less, the general arguments would hold for "real" cases.

In the case of grouting we can represent its efficiency in terms of the reduction in water make which its presence around the shaft will make. This is termed the Water Inflow Reduction Factor (WIRF), and is not synonymous with the permeability reduction of the grouted zone, for reasons which appear below.

We refer to Fig 1, where a shaft of radius r_2 is sunk through a formation of permeability k_1 with an undisturbed pore water pressure of P_θ. In the absence of grouting, the inflow per unit height of shaft Q_o (assuming continued flow and a recharge boundary distant R_o from the shaft centre, $R_o \geq\geq r_2$) will be given by:

$$Q_o = \frac{2\pi k_1 P_e}{\ln(R_o/r_2)} \tag{1}$$

In the presence of a grouted zone extending from r_2 to r_g where the permeability of the ground has been reduced to a value of k_g, the inflow Q_1 is given by:

$$Q_1 = \frac{2\pi k_1 P_e}{f \ln(r_g/r_2) + \ln(R_o/r_g)} \tag{2}$$

where $f = \frac{k_1}{k_g}$ = permeability reduction factor.

Thus the WIRF is given by:

$$WIRF = \frac{Q_o}{Q_1} = \frac{f \ln(r_g/r_2) + \ln(R_o/r_g)}{\ln(R_o/r_2)} \tag{3}$$

It is apparent from this equation that the WIRF must always be considerably lower than f. For instance, if we take a shaft of radius 4m surrounded by a grouted zone of thickness 3m and an undisturbed radius of 1000 m we have:

$$WIRF = 0.101 f + 0.899$$

To attain a, WIRF of 10 would require a value of f of 90.1. To achieve such a value of f in turn requires both a low permeability grout and exceedingly thorough grout treatment—at least 99% complete. Achieving such results can be very difficult and very time consuming, and sometimes not possible at all.

The reason for the very high standard of grouting required to achieve useful results can be seen if one examines the pore water pressure distribution both with and without a grout curtain.

Fig 1 shows that the "damming" effect of the grout curtain results in essentially full water pressure acting across the curtain. This is contrasted with the very reduced water pressure acting across the same zone when ungrouted.

Effectiveness of Wells

The use of wells to draw down an aquifer prior to excavation is a standard method of water control, particularly for shallow excavations. For deep shafts such systems have shortcomings both from the point of view of cost (pumping large quantities of water from deep aquifers requires submersible pumps and large amounts of power) and effectiveness.

For instance residual leakage can be high if the shaft penetrates an impermeable layer beneath the aquifer (as is usually the case). In such cases, the well pressure cannot be reduced below shaft pressure, with resulting leakage into the shaft. In

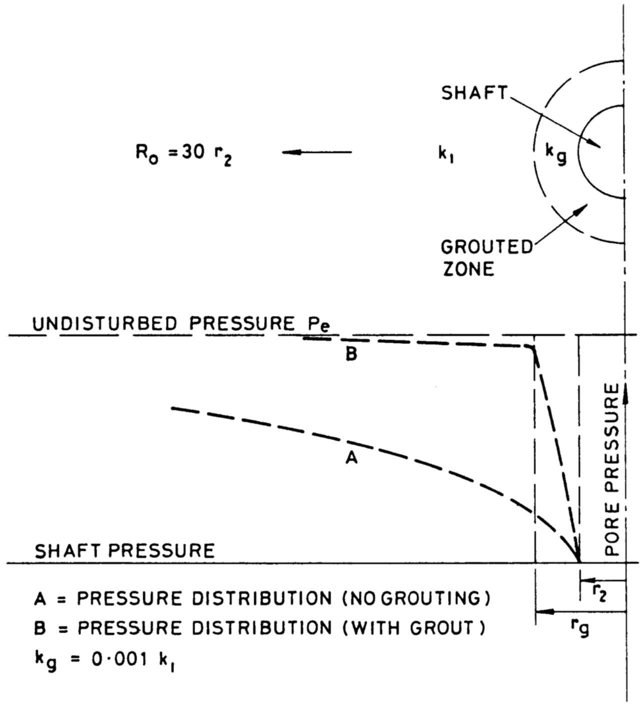


Fig 1 Effect of Grouting on Pressure Distribution

addition, drilling from the surface for the placing of submersible wells at depths of the order of 1,000 m is expensive, and the duty of the pumps in handling tens of litres/sec against such heads is very onerous.

Consequently, attention has turned to the possibilities of carrying out water control by wells installed from the shaft. Such wells, because of geometrical considerations, cannot easily be placed far from the excavation line. Because of this close proximity, it is necessary to use more wells than normal. Also, as will be shown below, for them to be really effective, well pressures must be lower than shaft pressure. Such conditions are difficult to achieve, and require either submersible pumps or down-the-hole ejector systems.

By applying the principle of superposition[1], the pressure p at any point in a system of flow in a confined aquifer which contains n wells can be written as:

$$p = c + \frac{1}{2\pi K} \sum_{1}^{n} Q_i \ln d_i \tag{4}$$

where Q_1 is the flow into the i^{th} well out of a total of n, and the distance of the well from the point in question is given by d_1. C is a constant depending upon the boundary conditions and which is eliminated during the mathematical development. The assumption is made that the well radii are small compared to all other dimensions in the system. Whilst this is not the case where the shaft constitutes one of the wells, the error resulting from such an assumption is in fact very small, as can be seen by comparison with Scott and Daw[2].

If we consider the situation as depicted in Fig. 2, with n wells of radius r_w positioned on a Pitched Circle Radius of D, the shaft of radius r_z at its centre and a distant boundary of constant pressure at the radius of R_o, we find that the flow Q_z to the shaft and flow Q_n to each well are related by the Water Inflow Reduction Factor (WIRF):

$$\text{WIRF} = \left[1 - \frac{n \ln(D/R_o)\{\ln(D/R_o) - m \ln(r_z/R_o)\}}{n(\ln D/R_o)^2 - \ln(r_z/R_o)\ln\{n(r_w/D)(D/R_o)^n\}}\right]^{-1} \tag{5}$$

where m is the ratio of drawdown at the wells to drawdown at the shaft. Such wells in a confined aquifer can be looked upon as effecting pressure relief, rather than aquifer drainage as would apply in the unconfined case.

In Fig 3, we have plotted WIRF against n over the range from 4 to 18 and for two values of r_w. It will be seen that for a realistic number of wells, drawdown at the well equal to that at the shaft results in a WIRF of 3–4, which is often not sufficient for proper water control. Further, any inefficiency in well drawdown, or well losses, have a marked effect on the WIRF such that the system becomes of marginal use.

The reason for such behaviour is shown in Fig 2, where it can be seen that the presence of the shaft reduces the effective head against which the wells can work.

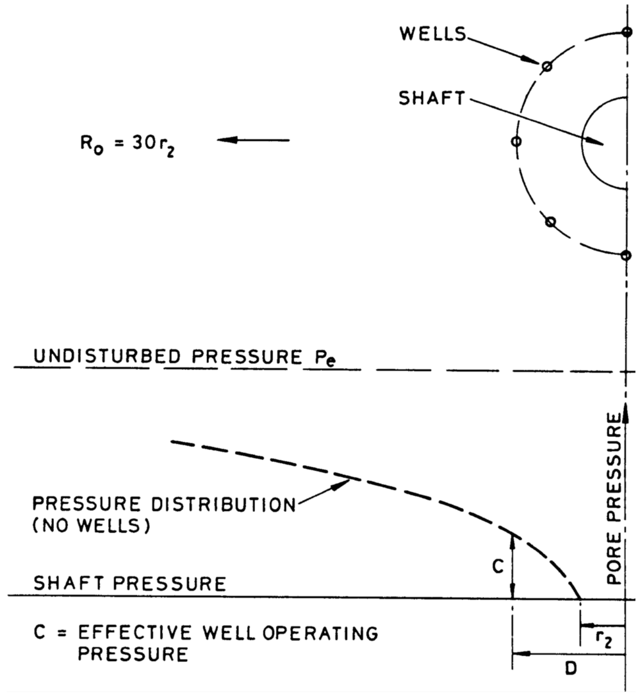


Fig 2 Effect of Shaft on Well Operation.

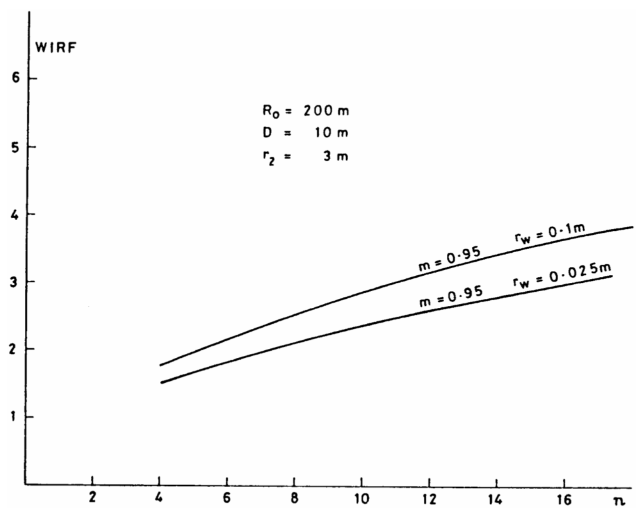


Fig 3 Effect of Number of Wells on WIRF.

Combined Grouting and Depressurising.

Grouting is not as effective as might be expected, because pore water pressure builds up behind the grout curtain; de-watering efficiency is reduced because of the reduced pore water pressure in the vicinity of the shaft.

Thus a combined system, where the wells can operate against a pressure increased by the grout curtain, and the grout curtain operate against a pressure reduced by the wells placed outside the grouted zone has obvious advantages.

Using the superposition approach similar to that used by Muskat[1], it is not possible to obtain a simple expression for WIRF. However, the following relationship allows calculation to be made:

$$m' = \left\{ \frac{1 - \left(\frac{Q_1}{Q_0}\right)\left(\frac{K_1}{K_g}\right) \frac{\ln(r_g/r_1)}{\ln(R_0/r_2)}}{\ln(r_g/R_0)} \right\} \left\{ \ln \frac{D}{R_0} - x \right\} \qquad (6)$$

where

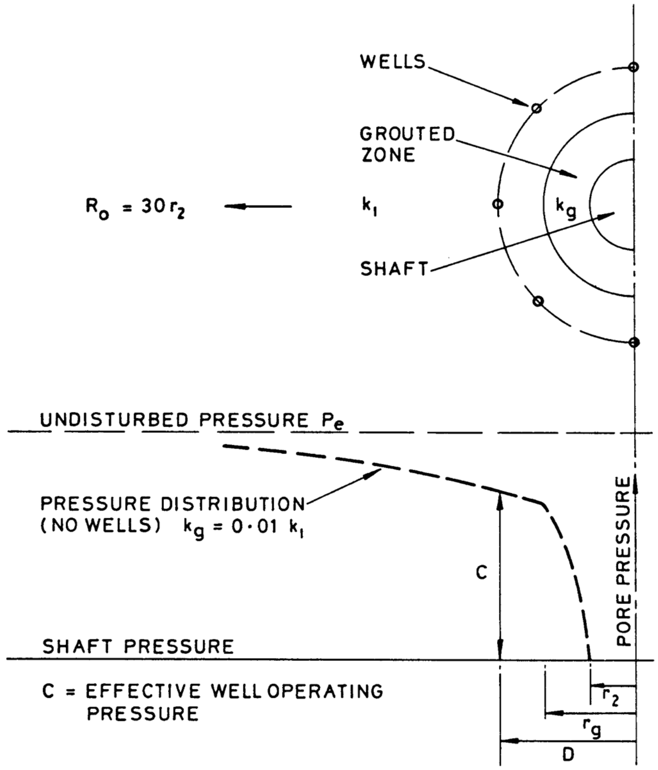


Fig 4 Effect of Grouting on Well Operation.

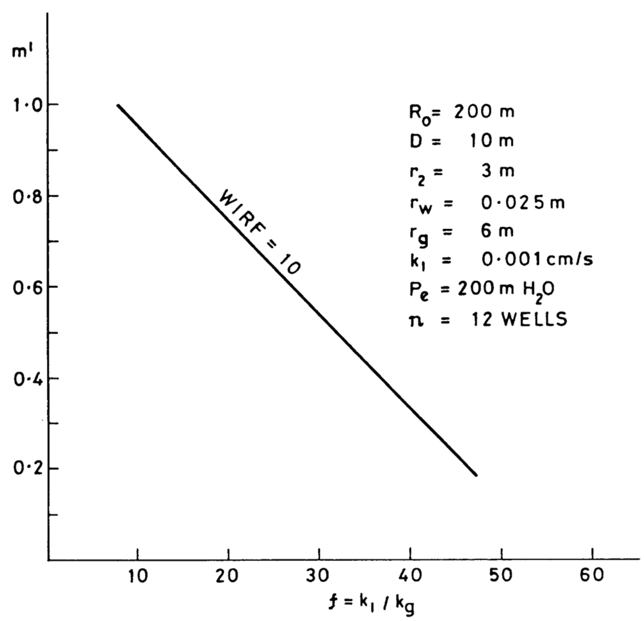


Fig 5 Interaction between well drawdown and permeability reduction.

$$x = \frac{\left\{n\left(\ln D/R_0\right)^2 - \ln r_2/R_0 \ln\left[n\left(r_w/D\right)\left(D/R_0\right)^n\right]\right\}\left\{1 - \frac{Q_1 \ln\left(r_g/R_0\right)}{Q_0 \ln\left(r_2/R_0\right)}\right\}}{n \ln\left(D/R_0\right)} \tag{7}$$

and m’ is the ratio of drawdown at the wells to drawdown at the shaft.

For the condition shown in Fig 4, we obtain for a WIRF of 10 a relationship between m’ and k_l/kg which is given in Fig 5. It will be seen that such a WIRF can be achieved with a relatively poor grout curtain (k_l/k_g=12,5) and inefficient depressurising well scheme (m'=0.9); on their own such grouting would give a WIRF of 2.9, and depressurising a WIRF of 1. 9.

It will be appreciated that the depressurising well system represents a diversion of the ground water from the sump to a part of the shaft where it can be properly collected and pumped to the surface. Such a system may still require considerable pumping capacity.

It is also possible to place the wells within the grouted zone. This obviously reduces the water to be pumped, but at the expense of reducing the well effectiveness.

We can use the method of Aravin[3], adapted for radial flow, to calculate the various pressures and flows under these conditions. The series of equations obtained is too long to detail here, but calculations made from them show that such a system tends to allow increased leakage into the shaft whilst reducing the volume of water to be pumped from the wells. For instance, in the example given in Fig 5, if the wells were placed in the centre of the grouted zone, the condition for a value of m' of 0.95 and f of 10 would be (figures with wells outside grouted zone in brackets):

WIRF = 4.57 (10.0)

Well inflow	0.89 lps per m (2.95 lps/m)
Shaft inflow	0.65 lps per m (0.30 lps/m)

Such methods should therefore be only considered where such a volume is critical.

CASE HISTORY REVIEW

Deep Wells

Both Greenslade[4] and Juvkam-Wold[5] describe the use of deep well depressurising systems in shaft sinking for uranium deposits in the Grants mineral belt of north west New Mexico, USA. The deposits lay at depths of between 600 and 1,200 m and are within and overlain by a sequence of very thick sedimentary deposits. These comprise alternating shales and sandstones with the sandstones representing major aquifers, as illustrated in the sequence described by Greenslade (Fig. 6), and where the lower Westwater Canyon Sandstone represents the actual ore horizon. The permeability of the sandstones is generally primary (i.e. intergranular, as opposed to fissure) and the aquifers are sometimes under artesian pressure.

Juvkam-Wold's paper contains some very useful design criteria which examine the effects of varying the number of wells, wellbore diameter, well circle radius and pumping time on aquifer pressure. The optimum location for the wells has to balance the need to be close to the shaft to get the maximum depressurisation, but sufficiently distant not to be significantly effected by the "cone of depression" of the shaft when excavation proceeds into the aquifer in question. The effect of wellbore size and pumping time is not so critical with the former being determined mainly by practical pump sizes and pumping effects being most significant during the first month.

Juvkam-Wold's "base case" comprised a ring of 10 wells at 270 m radius circle around the shaft with 305 mm well bore diameter, and he reports its practical application to depressurising the Westwater sandstone aquifer (950 m depth×60 m thick) at Gulf Minerals Mt Taylor Mine, near Ambrosia Lake, New Mexico.

Fig. 7 compares the measured aquifer pressure with the theoretical values and indicates a reduction to some 15% of the initial pressure, prior to the shaft excavation entering the aquifer. Peak production from the well system was about 4,500 gpm (280 l/s) and this then decreases steadily as the effects of the shaft-excavation "comes into play".

At the time the shaft enters the aquifer the well flow has decreased to about 150 l/s. After 250 days the total flow to the wells is equal to the total flow into the shaft/mine.

Juvkam-Wold claims that in this particular area they found depressurisation for water control to be more cost effective than grouting but he mentions cases where the two procedures should be used in conjunction.

Greenslade's paper describes such a combined treatment for sinking through the various aquifers indicated in Fig. 6. The design for the optimum system to cater for the differing requirements of the various aquifers was based on a detailed pumping trial with observation wells in different aquifers. When combining grouting with a depressurising scheme it is necessary to keep the groundwater flowrate, generated by the pumping below a certain level≤0.6 m/day is quoted) in order to restrict grout injection to the required zone around the shaft.

Fig. 8 shows the well layouts described by Greenslade for the six aquifer system with wells deepened to serve different aquifers, and with a separate four well pattern for the deepest Westwater Canyon sandstone. These well patterns are much closer to the shafts than the one described by Juvkam-Wold, but here the presence of grout curtains limit the drawdown effects of the shaft excavation and keep the well efficiency reasonably high throughout, as described previously in this paper.

DEPTH (feet)	GROUP / FORMATION	STRATIGRAPHIC UNIT	ROCK TYPE (% OF FORMATION)	TRANSMISSIVITY (m^2/s)	Nº OF WELLS	HEAD REDUCT. AT SHAFT (%)	AV. WELL PUMP RATES L/s.	SHAFT INFLOW L/s. W/OUT WELLS	SHAFT INFLOW L/s. WITH WELLS
	MESA VERDE GROUP	MENEFEE	SANDSTONE (30%) SILTSTONE (35%) SHALE (30%)						
500	MESA VERDE GROUP	POINT LOOKOUT	SANDSTONE (100%)	$2 \cdot 9 \times 10^{-4}$	3	77	9·5	49.2	12·6
	MESA VERDE GROUP	SATAN TONGUE	SILTSTONE (70%) SHALE (25%)						
1000	MESA VERDE GROUP	HOSTA TONGUE	SANDSTONE (95%)	$7 \cdot 2 \times 10^{-5}$	3	71	4·7	26·8	6·3
	MESA VERDE GROUP, CREVASSE CANYON FM.	MULATTO TONGUE	SANDSTONE (45%) SILTSTONE (20%) SHALE (35%)						
1500	MESA VERDE GROUP, CREVASSE CANYON FM.	DALTON	SANDSTONE (95%)	$1 \cdot 4 \times 10^{-5}$	3	73	1·9	10·1	3·2
	MESA VERDE GROUP, CREVASSE CANYON FM.	DILCO	SLST. SDS. SH. COAL						
	MESA VERDE GROUP	GALLUP	SANDSTONE (100%)	$1 \cdot 0 \times 10^{-4}$	3	73	6·3	49·2	15·8
2000–2500		MANCOS (MAIN BODY)	SHALE (85%) SILTSTONE (10%)						
		DAKOTA	SANDSTONE (80%)	$8 \cdot 6 \times 10^{-5}$	3	73	20·2	110	31·6
3000	MORRISON FORMATN.	BRUSHY BASIN	SHALE (90%)						
	MORRISON FORMATN.	WESTWATER CANYON	SANDSTONE (85%)	$1 \cdot 4 \times 10^{-4}$	4	79	25·3	158	34·7
	MORRISON FORMATN.	RECAPTURE	SANDSTONE (85%)						

Fig 6 Performance of depressurising system for single shaft (Greenslade 1979)

In Shaft Wells

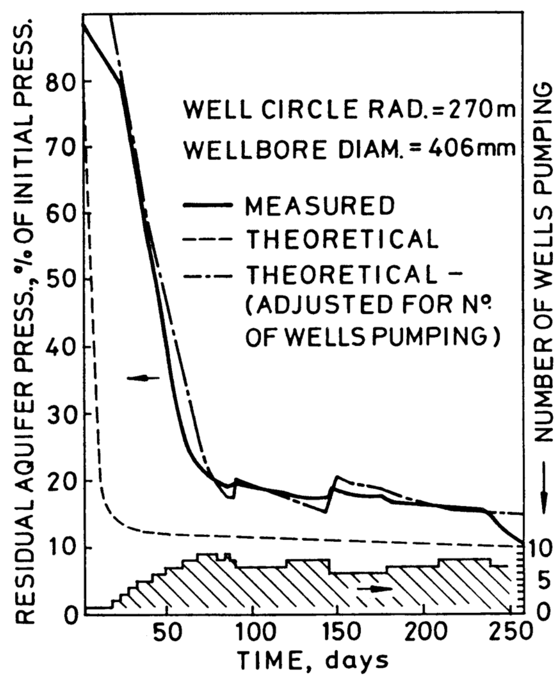
Fotheringham and Black[6] describe two applications of depressurising well systems during shaftsinking, where short wells are drilled from within the shaft excavation in a “cover”-type arrangement similar to that used for in-shaft grouting. These were employed during the sinking of the Riccall shafts at Selby new mine, Yorkshire, UK. Whilst. the aquifers were much thinner and potential groundwater inflows less than those described above, in the New Mexico Shafts, the requirements for groundwater pressure relief were also quite different.

In the first instance the shafts had to pass through a 5 m thick zone of weakly cemented Basal Permian Sands at a depth of about 430 m and the relief well system was used in combination with grouting (mainly chemical) to reduce the groundwater pressure and hence the likelihood of collapse of the sandstone under excavation. A first grout cover had been injected from a sump level of 400 m to contain groundwater within the lower section of the Lower Magnesian Limestone. Sinking then proceeded to a sump level of about 411 m from which the Basal Sands grout cover was injected. From the same level a ring of twelve equally spaced pressure relief wells were drilled at such an angle to take them outside the grouted zone, as indicated in Fig. 9(a).

Each hole was filled with a fine sand screen and their top ends were connected via a standpipe to a 110 mm diameter manifold system which discharged into a tank and was subsequently pumped by pipeline to a temporary pump lodge at the 398 m level. Initially the wells yielded some 26.6 l/s but this reduced to a steady 7.0 l/s within a day or so. At the completion of grouting in the second cover the total inflow from four central test holes within the Basal Sands was about 2.0 l/s, and this was subsequently reduced by installation of the well system to about 0.3 l/s.

In the second instance the shafts were faced with passing through some 15 m of the Woolley Edge Rock, a massive, medium strength sandstone at a depth of 650 m. The permeability was not particularly high and potential shaft inflow was of the order of 2.7 l/s. The main problem was one of depth and the subsequent difficulty in providing conventional plain concrete lined shafts to withstand the hydrostatic pressures. The use of pressure-relief wells in this situation enabled the backwall grouting to take place at injection pressures lower than would have normally been possible.

Due to the relatively low volumes of water present it was decided not to grout the Woolley Edge Rock so control of groundwater was entirely by the depressurising wells. A ring of eighteen such wells were drilled at a fan angle of 22° to the vertical from a specially constructed crib in the shaft wall at the 640 m level. The general arrangement for the depressurising system was similar to that used in the Basal Sands and is shown in Fig. 9(b). In the event both shafts entered the Woolley Edge Rock at about the same time and hence acted as additional dewatering sources for each other. Initial total well flow was 2.74 l/s but when steady conditions were obtained this reduced to only 0.76 l/s at the no. 1 shaft and 0.61 l/s at the no 2 shaft. Corresponding inflows to the shafts were 0.15 l/s and 0.23 l/s respectively. The concrete linings were subsequently cast



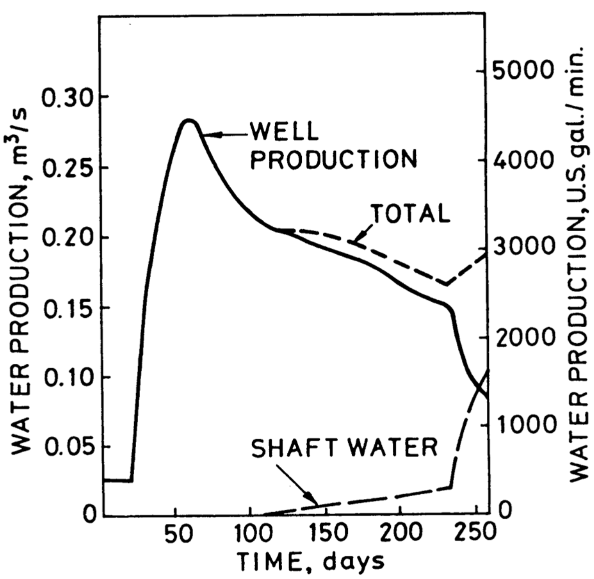


Fig 7 Performance of depressurising well system at Mt Taylor mine (Juvkam-Wold, 1982)

through the Woolley Edge Rock and at the time when the No 1 shaft sequence was backwall grouted flows naturally increased at the No 2 shaft to 0.91 l/s at the wells and 0.68 l/s from the shaft wall grout pipes.

Final backwall grouting was completed in two main passes at up to the pressure level of the aquifer instead of the normally used 1.25×hydrostatic.

CONCLUSIONS

A theoretical approach has been used to indicate how a combination of grouting and depressurising can be considerably more effective than either system of ground treatment when used separately, to reduce groundwater inflows during shaft sinking. Several aquifer conditions are suggested where such a combined treatment should prove beneficial and case histories are described from projects both in the USA and UK where the method has been employed. The well systems are either installed as deep wells drilled from surface or drilled as a 'cover' arrangement from within the shaft.

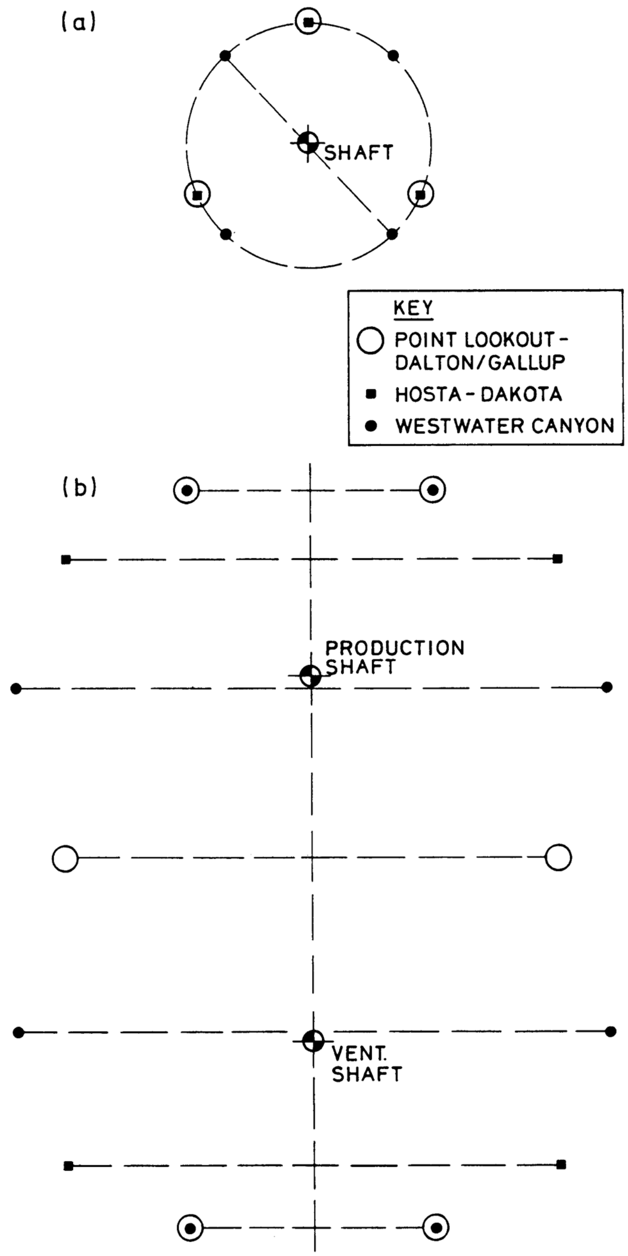


Fig 8 Depressurising well lay-outs for (a) single shaft, and (b) twin shaft, systems (Greenslade 1979)

Generally the method has had limited use, but it is hoped that it will be given more consideration in future shaft sinkings where the appropriate conditions prevail.

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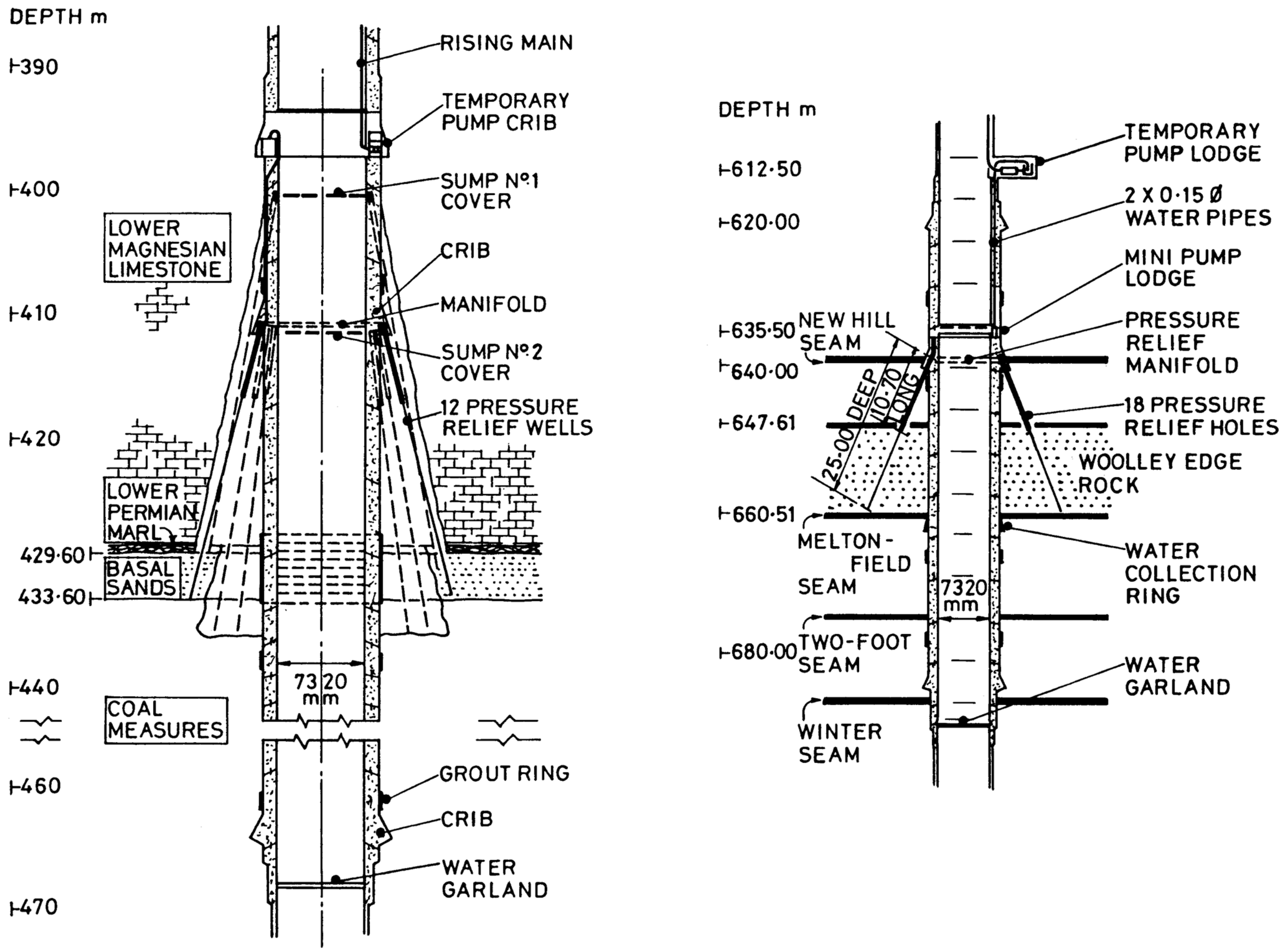
REFERENCES

1. Muskat, M, (1937), The Flow of Homogeneous Fluids through Porous Media, Chap. IX, publ. McGraw Hill.
2. Scott, R.A. and Daw, G.P. (1983), Ground Water Pressure Relief Wells in Shaft Sinking, Int.J. Mining Engng. Vol. 1 pp 229–236.
3. Aravin, V.I. ad Numerov, S.N. (1965), Theory of Fluid Flow in Undeformable Porous Media, Chap. 7, Para. 87, Israel Program for Scientific Translations, Jerusalem.



(a) Basal Permian Sands (b) Woolley Edge Rock

Fig 9 Arrangements of depressurising well systems at Riccall shafts, Selby (Fotheringham & Black, 1983)

4. Greenslade, W.M., (1979), Dewatering of Open Pit and Underground Coal Mines and Depressurisation during Shaft Sinking, Proc. 1st Int. Mine Drainage Symposium, Denver, Colorado, May 1979.
5. Juvkam-Wold, H.C., (1982), Dewatering of Ambrosia Lake Mines, Mining Engineering, September 1982, pp 1344–1350.
6. Fotheringham, J.B. and Black, J.C., (1983), Groundwater Pressure Relief in Shaft Sinking, The Mining Engineer, August 1983, pp 85–91.

Hydrogeological investigations and assessments for shaft sinking

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SUMMARY

The paper is a critical review of borehole investigation techniques currently used to acquire the hydrogeological data necessary to design and construct a new mine shaft. Particular reference is given to shaft sinking through water bearing strata typical of the conditions experienced in the UK coalfields.

Aspects covered include core drilling, geological and geotechnical logging of recovered core material, laboratory testing of soil, rock and groundwater samples, wireline geophysical logging, hydrological testing and in-situ stress measurement.

The importance is stressed for good correlation and assessment of the data in terms of determining the nature of the aquifer permeability, the use of various aquifer modelling methods and the prediction of likely groundwater inflows to the developed shaft. Also discussed is the use of the data for selection of the optimum method of ground treatment, for design of the shaft linings and for choice of the most appropriate method for excavation of the shaft.

In addition some indication is given of the approach to the organisation and a guide to current costs for such a shaft investigation programme.

INTRODUCTION

The current economic climate for the development of a new mine, or for accessing new reserves within a current mine, by the construction of shafts, is such that the shaft construction cycle is on the critical path for overall mine development and hence early return on investment. It is essential therefore that the hydrogeological and geotechnical conditions for shaft sinking are identified accurately at an early stage to enable the most efficient and cost-effective methods of shaft construction and ground treatment to be designed and implemented and to ensure that the sinking timetable can be maintained.

A thorough site investigation study is therefore required and this involves both a detailed review of pre-existing information for the area together with a fully tested borehole on the site of the proposed shaft.

The subsequent compilation of all the derived data serves five main purposes:

(i) it provides the client with initial design data
(ii) it enables the client to prepare tender documents
(iii) it provides potential contractors with data on which to base ground treatment and shaft designs, (if not specified), or to offer alternative designs.
(iv) it enables the client to assess the inherent risks attached to the above, equating individual and total costs in terms of time and finance required.
(v) it provides the chosen contractor and client with a permanent source of reference during the shaft construction programme.

The main intention of this paper is to look critically at several different approaches to shaft investigations which are dictated in particular by conditions existing in the UK coalfields. The various test categories and current techniques which are considered most appropriate for the different circumstances are described and some guideline indications are given for the costs of various elements of the investigation programme in order to put them into perspective with the overall cost of shaft sinking and mine development.

GENERAL SPECIFICATION FOR ENGINEERING BOREHOLES

Desk Study

The first phase of any new shaft development should be a desk study whose purpose is to correlate all relevant existing information. Sources of information might include existing borehole data, old shaft sinking records, adjacent workings, maps and plans and finally outside agencies such as water authorities and the British Geological Survey. This early study will not necessarily be particularly site specific.

Position of Engineering Borehole

Site specific data will involve the drilling of an engineering borehole either adjacent to or within the proposed shaft section.

For deep boreholes, in excess of 1,000 m, in dipping strata, it can prove difficult and very expensive to maintain a borehole within a 3 to 4 m radius, and therefore a degree of tolerance is required.

The principal areas where the site specific data is required are:

(i) to ascertain the position and nature of any potential faulting likely to be encountered by the shaft.
(ii) to provide precise levels for inset and pit bottom drivages.
(iii) to determine aquifer thicknesses where bed boundaries are not planar eg Basal Sands Barchan Dune Structure.

In certain circumstances where, for example, surface casing has been abandoned or a shaft centre borehole poorly abandoned the location of the engineering borehole within the shaft section can be a positive disadvantage. Such a poorly cemented borehole within a series of Upper Coal Measure sandstones at the North Selby shaft was thought to be contributing up to 2.3 l/s into the shaft sump at a higher level through an aquitard.

In areas free from major faulting and in well bedded sedimentary strata an engineering borehole within 10 m of the proposed shaft should adequately describe potential shaft sinking conditions. In areas of dipping strata engineering boreholes should, if possible, be positioned on the down dip side of the shaft to allow for up dip migration.

Drilling Practice

The responsibility for all drilling operations including hole stability, hole deviation and core quality normally rests with the drilling contractor or an appointed consultant. At tender stage, the drilling specification composed by the client should outline a series of criteria that clearly describe the following drilling parameters:

(i) Core Quality: Minimum core recovery should be 99% in any 10 m. Core quality is also of prime importance; it is essential the core should be free of drilling induced fractures. The penalty for unacceptable core loss or poor quality will be a diversion.
(ii) Hole Stability: To facilitate good geophysical logs and downhole hydrological packer seats it is very important to have an "on gauge" hole. If packer testing is to be conducted and excessive over break can be attributable to bad drilling practice, provision should be made for the hole to be reamed out, to enable larger packers to be used.
(iii) Hole Deviation: Maximum permitted deviation should be 5° for engineering boreholes.
(iv) Mud Control: Mud control is either the responsibility of an independent consultant or a drilling contractor, and should be "tailored" directly to the hydrogeological conditions that may exist downhole. If high mud losses do occur during the drilling, the consequences of using lost circulation material, casing, or cement stabilisation to regain control of the well have to be fully discussed at all times with the client.
(v) Core Handling: A detailed method statement should be included in the drilling specification that controls how the core should be removed from the core barrel, how it is cleaned and how it should be stored and racked.
(vi) Drilling Records: Where possible a comprehensive series of drilling records should be maintained throughout the duration of the borehole. This should include: drill weight, mud details, mud pressure, hole velocity, mud losses, penetration rate and gas emissions. This service can be conducted by a specialist mud logging company.

Core Logging

As the cores will provide a permanent source of reference during the shaft sinking operation, when possible, core description should be strictly visual and the core left unbroken. Once the core is measured, it should be photographed and logged as soon as possible to record the saturated condition of the core before the argillaceous rock types dry out and disc. Generally all core should be processed within 24 hours of being drilled. Detailed descriptions should be made of all fractures and inherent planes of weakness and these should be categorised on their aperture, rugosity and nature. Core quality logs should include core loss, RQD and fracture index logs. On site logging procedures within British Coal include data capture directly onto a computer within the core shed, thereby allowing the immediate production of draft engineering logs and graphic sections.

In some circumstances on-site core testing may be appropriate. This could involve point load testing although its use as a general rock strength index should be regarded with some caution as testing of the less competent zones may not be possible. If any soft sediments are present, such as clays within a sandstone sequence, then standard soil classification tests can be useful.

Once the borehole has been completed it is essential to store the core on the shaft site to be readily accessible by all interested parties for reference.

Sample Selection

The question of the amount of core testing required reflects the need for a flexible approach and in particular for the involvement of a geotechnical engineer with a clear knowledge of the likely problems of shaft sinking through the different strata. In some cases the required methods of construction may not be very clear at the early stage in which case it may be appropriate, assuming the core is well preserved, to adopt a two phase approach to testing, with an initial package of fairly standard testing and only incorporating some of the more specialised (and usually more expensive) tests when various aspects of construction design become more apparent.

The sampling frequency is therefore a function of the method of excavation, lining design and ground treatment and should not be restricted to a rigid spacing of samples per metre.

It is far more important to obtain one sample from a bed that can be classified as typical, as compared to two or three at regular intervals that may be affected by bed boundaries etc. Absolute values of compressive strength and shear strength are a function of moisture content and saturation. For arenaceous samples any lost moisture can be replaced in the laboratory, however, for argillaceous rock types, once they have lost their moisture content they will tend to slake and break down if a further attempt is made to resaturate the core in an unconfined state. It is very important therefore that samples are selected and sealed as soon as possible. Moisture content can be retained in samples using wax, cling film or hot strippable plastics.

Core Testing

Once the samples are selected, a testing schedule must be drawn up that will quantify the laboratory programme. The design of the programme will reflect similar criteria described above on sample selection. Table 1 describes the range of tests applicable to a shaft sinking operation and indicates the principal areas in which the test parameter is used. The laboratory determination of geomechanical and hydrogeological parameters is a very expensive and time consuming operation. A typical testing schedule for a fully cored shaft centre hole down to 600 m could take up to 15 weeks to complete and would cost at least £60,000. It is therefore very important to select the correct samples and tests that are fully representative of their lithology and supplement the gaps with index logs from geophysical techniques.

Geophysical Logging

Geophysical methods for the determination of rock mass characteristics are an essential part of any site investigation programme[1]. Supplementary data from continuous geophysics logs can be used to “fill in” porosity and permeability data and rock strength data from laboratory tests. Caliper logs quantify swelling and slaking indices together with resistivity logs are very useful when selecting sites for hydrological tests. (See later section).

Recent advances within British Coal in the development of downhole seismic techniques such as inverted, multi-offset vertical seismic profiling IMOVSP as shown in Fig 1 can extend the area of investigation into the rock mass from centimetres to tens of metres. This is especially useful to confirm the hade and throw of faults etc that may or may not have intersected the borehole.

A table describing the principal downhole geophysical logs important to a shaft sinking operation is presented as Table 2.

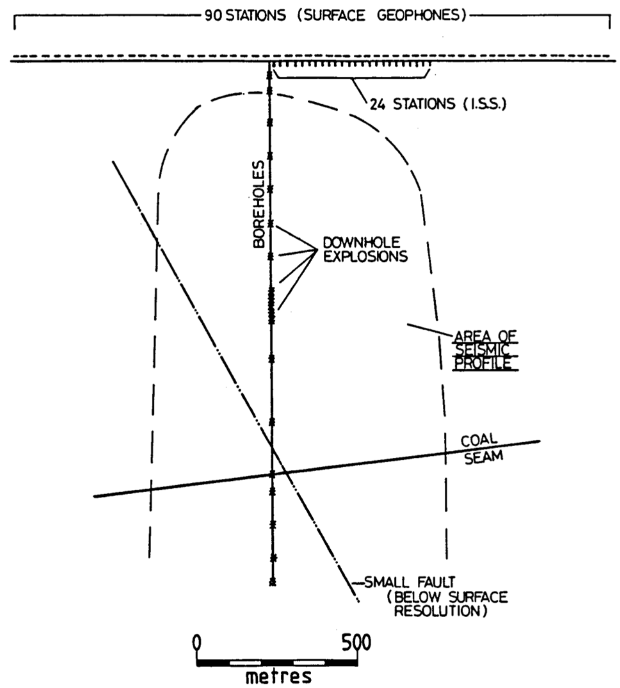


Fig 1: Inverse Multi-Offset Vertical Seismic Profile.

THE AQUIFER CASE

General

The previous section gives an indication of the minimum programme of testing and assessment that is required when relatively good, competent and dry conditions exist over the proposed shaft length. The situation is quite different when groundwater is present, and the basic information needs to be supplemented in certain areas to enable design of both the necessary ground treatment and special requirements for shaft linings.

Even when aquifers are known to be present the testing programme can be quite different depending on their disposition, frequency, thickness and depth. For example, the known occurrence of a thick and continuous aquifer from surface level, such as encountered with the Bunter Sandstone at the Selby shafts, can predetermine the method of ground treatment (i.e in this instance ground freezing), and only some limited testing within the aquifer zone may be necessary. In contrast, the presence of thinner, but heavily waterbearing aquifers, at depth e.g. Basal Sands and Coal Measures Sandstones at Selby, Sherwood (Bunter) Sandstone at Asfordby, can present different options for ground treatment with the optimum method only being selected following a detailed testing programme and subsequent assessment. The testing programme therefore needs to be diverse to provide the data to cover all the options. In the following subsections the aquifer situation is examined in some detail and the additional testing requirements introduced.

Categorisation of Aquifers

It is useful to attempt to classify aquifers such as those encountered in the UK coalfields and a suggested approach is based on the nature of the permeability and the rock strength (using laboratory measured unconfined compressive strengths (UCS) for initial assessment).

Table 1 Laboratory Test Schedule for Shaft Sinking

TEST TYPE	PRINCIPAL AREA OF USE	AVERAGE ROCK TYPE
Unconfined Comp. Strength	The most commonly used strength index. Essential for lining and inset design, excavation characteristics and ground stability.	All rock types
Unconfined Tensile Strength	Very difficult test to measure an absolute value. Used most in determination of excavation characteristics.	All rock types

TEST TYPE	PRINCIPAL AREA OF USE	AVERAGE ROCK TYPE
Triaxial Strength	Essential test for determination of deformation characteristics inset and lining design.	All rock types Critical for very weak rock types.
Elastic Properties Youngs Modulus Poisson Ratio	Very important parameter for determination of deformation characteristics for inset and lining design. Frequency of sampling less than for UCS and Triaxial.	All rock types
Swelling Strength Slake Durability	Important test for argillaceous rock types—Essential for excavation characteristics and for shaft drilling.	Argillaceous rock types ie mudstones
Moisture Content	Very important test to calibrate laboratory rock strength data.	All samples
Rebound Hardness Abrasivity Specific Energy	Very important parameters for rock cutting and shaft drilling	Stronger rock types
Permeability Porosity Bulk Density Thermal Tests	For ground treatment assessments i.e. grouting, freezing, dewatering	Permeable strata

Table 2 Geophysical Logging for Shaft Sinking Purposes

LOG TYPE	BRIEF DESCRIPTION AND MAIN APPLICATION
Caliper (1 arm, 4 arm)	Measures borehole roughness and rugosity—symptomatic of lithology and strength. Stress directions computed from orientated 4 arm (Break out log).
Verticality	Verticality measured by gyroscope accelerometers or a combination of a compass and a pendulum or strain gauge. Determines position of borehole.
Temperature	Sonde measures heat transfer between strata and mud column to assess temperature gradient etc. Important log for determination of ice wall design and mining environment.
Spontaneous Potential	Recording made with reference to earth electrode at ground level. Detects mudstone against sandstone boundaries.
Focussed Electric	A point resistance electrode with two extra guard electrodes to focus the direct current into formation: improves penetration and resolution.
Microlaterlog	A pad mounted FE tool with high resolution to assess salinity changes due to mud invasion. Very useful tool for measuring water quality and assessing possible zones of porosity and permeability.
Dipmeter	Computed correlation log using 3, 4 or 6 microlaterolog traces give values of local dip, fault hade etc. Computed logs include vertical fracture frequency and breakout logs often run in conjunction with Verticality tools.
Induction Logs (Deep Induction, Medium Induction, Shallow Induction)	Coils housed in the body of the sonde induce detected alternating currents in the formation. Record gives formation conductivity for use in water saturation calculation. Penetration of logs picks up changes in conductivity due to mud invasion. Very useful in assessing zones of porosity and permeability.
Gamma Ray	Natural radioactivity from absorbed ions, usually related to clay mineral content. A lithology log least affected by caving.
Natural Gamma	Larger detectors count number of gamma rays having energies corresponding to potassium, thorium and uranium. Cross plots allow determination of clay mineralogy and evaporites.
Density Logs (Long Spaced Density High Resolution Density Bed Resolution Density)	Tools have gamma source and gamma detector. Provides measure of electron density proportional to bulk density. Assumed matrix density allows direct display of porosity.
Neutron Neutron	Tool provided with a fast Neutron source and a slow neutron detector: provides a measure of light nuclear density, essentially

LOG TYPE	BRIEF DESCRIPTION AND MAIN APPLICATION
	hydrogen content therefore relates to water content. Log can be calibrated, scaled and displayed as neutron porosity. Hydrogen index also taken for low porosity rocks to indicate micro fracture frequency. Computed RocTec Log calibrated in terms of rock strength.
Sonic Log or Sonic Delta t	Travel time recorded in micro-seconds/unit length between transmitter and receivers.
Multi Channel Sonic	Sonic Tool with 3 or 4 receivers and transmitters, very good for caved holes. Interval velocity a function of rock strength and rock matrix porosity. When sonic porosity is cross plotted against Neutron or Density porosity (ie total) a measure of fracture porosity is possible.
Survey Ref Inverse multi— offset Vertical Seismic Profile	String of geophones are extended along the surface and an energy source downhole. With one or more sources located away from the borehole. 3D VSP's may be shot, giving position strike, and dip of local faults etc.

TABLE 3:- Categories of Aquifer

AQUIFER CLASSIFICATION	NATURE OF PERMEABILITY	ROCK STRENGTH	TYPICAL AQUIFER IN UK COALFIELDS
A	SECONDARY	HIGH	LOWER MAGNESIAN LIMESTONE (WISTOW)
		>50 MPa	
B	COMBINED SECONDARY & PRIMARY	INTERMEDIATE	PASSAGE GROUP SANDSTONE (CASTLEBRIDGE)
		<20 MPa	
C	PRIMARY	LOW	SHERWOOD SANDSTONE (ASFORDBY)

At the opposite extremes are high strength rock with entirely secondary permeability i.e fissures, vugs etc, and very low strength rock with entirely primary permeability i.e intergranular "pores". Such a breakdown is not strictly accurate as all rock will possess an intergranular permeability however low, and is likely to contain some content of fissuring, however fine.

In Table 3 three main categories of aquifer are given together with a tentative suggestion for ranges of rock strength. Quantitative values of permeability are not included in the same way since it is the nature of the permeability which will determine the forms of testing required and ultimately the method of ground treatment. Examples of known aquifers encountered in previous shaft sinkings, which fit best within these three categories, are also included in Table 3. Such a classif ication cannot be considered as rigid and several subdivisions could be included within each category with a general transition throughout the sequence.

Another factor of significance is the depth of the aquifer. For example a weak permeable category C sandstone may still be reasonably competent in excavation at shallow depths but will be a completely different proposition at greater depths under the higher groundwater pressure and potential inflow.

The Cored Borehole

As with the base case a fully cored borehole on or near the shaft centreline is an essential feature of the investigation programme. Full core recovery may be more difficult in this instance particularly if very weak waterbearing sands are encountered, but every effort should be made to obtain good core from these most critical zones for the shaftsinking. The need for careful handling and good conditions of storage and access for the core at all stages, until at least the completion of shaft construction, cannot be over emphasised.

Drillers Log

Again this will follow the requirements for the base case except that considerable value can be attached to a detailed record of drill fluid loss zones and water level measurements in identifying the more permeable horizons. Together with a careful inspection of the core this can aid considerably in the selection of the zones which require more detailed testing.

Core Logging

The standard "package" of core logs for the base case should be followed, but with even more emphasis on getting a complete record. In the geological and geotechnical logs the more open fractures and obviously permeable zones should be highlighted.

Core Testing

In this category there is probably the greatest divergence from the base case with the testing programme very much dictated by the particular ground conditions. A certain standard package of tests will generally always be adopted and these will be supplemented as necessary.

For ground treatment requirements, and in particular grouting assessment, tests such as porosity and permeability should be carried out in the permeable granular zones. In specific cases where extensive chemical grouting may be required the tests may be supplemented by pore size distribution measurements.

Similarly when ground freezing is considered the likely option special tests such as thermal conductivity and specific heat can be invaluable as there is usually a lack of such data available for the ground freezing designer on the particular ground materials in question. Compressive strength tests on both unfrozen and frozen samples have also been used to assess the improvement in strength for less competent material.

Geophysical Logging

The basic package of logs listed in Table 2 is equally applicable although some distinction should probably be made between the requirements of the client and likely requirements of the shaft sinking contractor in this context. In the latter case the most useful logs are the caliper and temperature and secondly those logs that indicate the general presence of more permeable zones, such as laterolog and neutron log.

Hydrological Testing

Hydrological tests represent the single most important category of testing in the overall assessment programme for the "aquifer situation". They enable

- identification of the most permeable aquifer zones.
- measurement of the transmissivity and average permeability of these zones
- measurement of the aquifer pressure
- analysis of groundwater samples from the aquifer.

The choice of test zones should be the forte of the hydrogeologist who will correlate the results of core logging and geophysical logging with a close inspection of the core material in order to produce the optimum test programme. The actual method of testing is very important and the area of hydrological testing for shaft sinking has been discussed by the authors previously[2,3].

In essence the most appropriate technique in nearly all instances is one in which a period of groundwater flow is promoted from the test zone, followed by a period of shut-in when the flow is stopped and the groundwater pressure in the zone is allowed to recover. Analysis of the pressure records enable the aquifer parameters to be defined. Such a method is based very much on the oil industry drill stem test which can be used effectively in certain aquifer conditions, but requires considerable modification to cope with the heavy inflows encountered in many UK coalfield aquifers. Such modifications have been carried out in recent years in the British Coal exploration programme and are described elsewhere[4].

Other developments, along similar lines and particularly appropriate in certain aquifer situations, have involved incorporating a downhole pump in the system[2].

Several other test methods, including injection tests between packers and borehole flow metering have been used with some limited success but are probably best considered as supplementary measurements to add resolution within a coarse test zone, particularly if specific fracture systems can be identified.

This area of hydrological testing is a continually developing field and represents a costly element in the overall testing package. Therefore, it is essential to employ the most appropriate test programme in each instance. Reliable results enable more confidence in the subsequent aquifer modelling and hence in the predictions made for shaft construction.

The measurement of in situ stress by the hydrofracturing method[6] has been introduced fairly recently by British Coal to their engineering borehole programmes and the technique uses essentially the same "down-the-hole" packer equipment as that employed for the hydrological testing.

Data Assessment and Design Aspects

Having accumulated data from the various sources it is the overall correlation and assessment that produces the necessary design parameters for ground treatment, shaft lining and method of excavation.

The hydrological testing will have highlighted the critical zones and given overall magnitudes of likely inflow thus indicating where ground treatment will be most necessary. The next phase is the assessment of the nature of the aquifer, as generally categorised in Table 3, and this will be based specifically on correlation of geotechnical and geophysical logs with' laboratory test measurements and a very careful inspection of the core from the aquifer zones.

Aquifer modelling methods are then employed to provide finer resolution of inflows within the individual aquifer zones and a recent publication by one of the authors indicates the current approach used by British Coal in this area[5].

The overall depth of the proposed shaft and the disposition of the aquifers within this depth are major factors in determining the options for ground treatment. For example, whilst freezing may be considered the obvious choice if a thick continuous aquifer is met from ground surface level, it may not represent the most cost-effective approach if a similar aquifer is encountered towards the base of a very deep shaft.

In instances where grouting is considered appropriate the nature of the permeability is critical. The presence, size and frequency of fractures, and the average pore size of the intergranular zones will determine the choice of cement and/or chemical grout.

Where chemical grouting is essential then it is necessary to further define the size of openings to decide whether a relatively 'coarse' chemical grout can be used or whether a highly clarified material is necessary e.g as at North Selby Shafts where a high speed centrifuge was employed to produce an ultrafine silicate grout for injection of the Coal Measures Sandstones. Temperature and chemical analysis of the groundwater within the aquifer zones are also major influences on grout formulation design. Another aspect is whether the aquifer material requires additional strengthening to enable safe excavation. Most chemical grouts will only provide minimal additional strength and if the aquifer is under high hydrostatic pressure benefits can be obtained by combining a groundwater depressurising system with the grout injection e.g as at Wistow & Riccall shafts, Selby Mine[7,8].

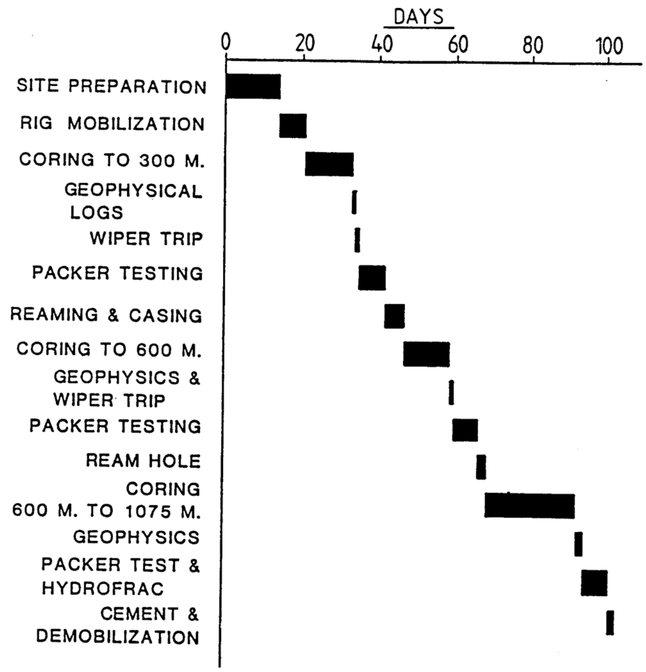
In the case of ground freezing the nature of the permeability is not so important, nor is the actual magnitude of potential inflow. Critical factors are the quality and temperature of the groundwater and the thermal properties of the rock or soil within the aquifer. The mobility of the groundwater can be a restriction on the use of ground freezing if for instance high potential gradients exist with resulting zones of high flow rate, which for instance might exist in highly solutioned rock or weathered zones at relatively shallow depth. Also the nature of the rock or alluvial material can present problems for drilling where a highly directional hole pattern is essential for efficient ground freezing.

Deep well dewatering may offer an alternative approach in some instances and here the main parameters of importance are the nature, permeability and depths of the aquifers and the the groundwater chemistry. The latter is obviously important from the point of view of disposal of large volumes of groundwater. Generally however, depressurising well systems in combination with other ground treatment, such as grouting, will be more appropriate. Client specifications for pumping limits from within the shaft can also dictate the ground treatment methods. Whilst ground freezing can be considered as a total exclusion method, grouting and dewatering or depressurising well systems will always leave some residual inflow to the shaft.

The requirements for shaft lining design will be very much aligned to the method of ground treatment adopted and the depth of the aquifer zones. Linings designed to resist the hydrostatic pressure will be necessary whatever method of ground treatment is employed and the potential use for higher strength concrete linings at greater depths is described in another current paper[9].

DURATION, COSTS AND COMMERCIAL RISK

A typical programme illustrating the phasing of a shaft centre hole is given in Fig 2. The bar chart describes a site investigation programme for a 1075 m deep shaft, whose upper 600 m consist of water bearing strata. The total duration of the drilling project is approximately 100 days. A breakdown of the time includes up to 3 weeks for hydrological testing and hydrofracture analysis. A very approximate breakdown in costs is also given in Fig 2. Whilst drilling costs form a significant portion of the total costs (43%) a considerable proportion of the total costs is taken up with the laboratory testing. The



COSTS	£	
DRILLING	200 000	
PACKER TESTS HYDROFRACING AND RIG TIME	120 000	
GEOPHYSICS	25 000	
SITE PREPARATION	20 000	
LABORATORY TESTING	100 000	
	465 000	TOTAL

Fig 2: Typical Programme for a 1075 m deep shaft borehole.

completion of the laboratory testing programme, depending on the magnitude and complexity of the test schedule and resources of the laboratory, could extend the site investigation programme by a further 15 weeks.

Although the total estimated cost of just over £450,000 is a substantial investment, the costs of the site investigation must be considered in light of the likely final shaft sinking costs. For a 7.5 m shaft to a depth of 1075 m, this could very easily extend up to £20,000,000. Therefore £450,000 site investigation programme would equate to approximately 2.25% of total costs. This is an acceptable percentage figure, providing the data quantify the degree of commercial risk and enable a detailed shaft design and tender specification to be constructed.

A study of the recent shaft sinkings operations at Selby has revealed that between 1.0 and 1.3% of total shaft sinking costs were spent on site investigation. This figure is significantly lower than the example given in Fig 2.

One reason for this low figure can be attributed to the fact that all of the laboratory rock testing was conducted “in-house” and therefore not charged to the budget. The site investigation programme at Selby consisted essentially of a fully cored shaft centre borehole either close to or down the centre of each shaft. The degree of hydrological testing varied from site to site and at the deeper levels, in excess of 400 m, was confined entirely to drill stem tests.

The success of the drill stem tests was rather varied: good results were obtained in the more competent horizons, but severe difficulty was encountered when testing very weak beds of Upper Coal Measures sandstones. The lack of well defined hydrogeological data for the weaker sandstone aquifers at Riccall and North Selby has been highlighted as being one of the factors responsible for significant delays when sinking through geological horizons of this type.

Adequate time should be made available to enable all the results from the site investigation to be analysed, collated and incorporated correctly into the relevant tender documents. Information available after the tendering stage may be technically very useful but very often contractually unproductive. When preparing geotechnical data for tender purposes it is very important to make a distinction between what is fact i.e. measured and what is interpreted.

Any geotechnical documents issued as part of the tender documents must be as factual as possible. If interpretations from borehole to a shaft for example are difficult to make, then they should not be included in the tender documents; only the factual recorded data should be included and the contractor asked to make up his own opinion and therefore share the commercial risk. This type of problem is illustrated particularly well when preparing contract documents for grouting. The success of a grouting exercise is not just dependent on measuring and interpreting the hydrogeological characteristics of the aquifer but is equally dependent on the skill of the contractor in the design and implementation of the injection programme. For the client in this instance to measure, fully interpret and to dictate to the contractor the pattern of holes, grout types and injection procedures etc would remove all the commercial risk from the contractor and place it squarely with the client. A far more sensible approach would be to present the basic data to the contractor, outline very broadly the preferred method of grout treatment and ask for the contractors' own detailed proposals that, if accepted, would be strictly adhered to during construction. In this way a contractor must accept a proportion of the risk, and his tender is appraised on technical as well as commercial grounds. Furthermore, it will be in the tender appraisal that interpretations made by clients' engineers will be compared to those of the contractors.

CONCLUSIONS

A well designed programme of site investigation is essential if future major shaft sinking operations are to run on time and within budget. Unfortunately, there are no simple criteria that can be used to quantify the level of investigation. This is very much dependent on the particular requirements for individual contracts, and needs to be "tailored" to actual ground conditions anticipated and methods of construction and ground treatment envisaged or predetermined from the preliminary review of exploratory boreholes or previous sinkings in the area.

For example the extent of testing required, when the full length of the shaft is known to be in competent dry rock and with no ground treatment required, can be considerably less than the instance when the shaft has to pass through a multi-aquifer system containing zones of potentially unstable ground. In the former case shaft lining design requirements may be minimal whereas in the latter case data may be required for the design of fairly complex shaft linings to counter high hydrostatic pressures at depth, and for the selection and detailed design of the most appropriate method of ground treatment. If there is is not a straightforward choice at this stage then a wide range of testing will be essential to cover the differing requirements for either grouting, freezing or depressurising.

The requirement is therefore for an adaptable approach to the design of hydrogeological investigations for shaft sinking and for this reason an experienced mining geologist, hydrogeologist or geotechnical engineer with a knowledge of shaft sinking requirements should be involved from the outset and in all subsequent phases of planning including the correlation, assessment and interpretation of the accumulated data. Only in this way can the most cost effective package be put together.

Furthermore, emphasis must be placed on how the information is to be used and in what format and context it is to be presented to the contractor. In this paper an attempt has been made to illustrate broad categories of approach that can be undertaken.

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REFERENCES

1. Atkinson, T., Dow, R. and Brom R.W.C. (1984). A review of hydrological investigations for deep coal mines with special reference to petrophysical methods, Int. J. Mine Water 3 (3)pp 19–34.
2. Daw, G.P. (1984), Application of aquifer testing to deep shaft investigations, Q.J. Eng. Geol., London 17 (4) pp 367–379.
3. Lloyd, J.W. and Jeffery, R.I. (1983). Deep aquifer testing methods and data interpretation, Z.dt.geol. Ges. 134 pp 871–884.
4. Daw, G.P., Fear, N.J., Jeffery, R.I. and Pollard, C.A. (1988). Hydrogeological investigations and ground treatment for shaft sinking at Asfordby new mine. Proc. 3rd Int. Mine Water Cong., Melbourne, pp 683–692.
5. Jeffery, R.I., Lloyd, J.W. and Edwards, M. (1989). The assessment of shaft inflow characteristics for deep aquifers associated with coal mining in the UK. Paper submitted for Int. Conf. on Shaft Engineering, Harrogate, June 1989.
6. Rummel, F, and Baumgärtner (1985). Hydraulic fracturing, in-situ stress, and permeability measurements in the Research Borehole Konzen, Hohes Venn (W.Germany), N.Jb.Geol.Palaönt Abh 171 pp 183–193.
7. Fotheringham, J.B. and Black, J.C. (1983). Groundwater pressure relief in shaft sinking, The Mining Engineer, August, pp 85–91.
8. Hutchinson, M.T. and Daw, G.P. (1989). Combined grouting, and depressurising for water control during shaft sinking. Paper submitted for Int. Conf. on Shaft Engineering, Harrogate, June 1989.
9. Auld, F.A. (1989). High strength, superior durability, concrete shaft linings. Paper submitted for Int. Conf. on Shaft Engineering, Harrogate, June 1989.

Assessment of shaft inflow characteristics for deep aquifers associated with coal mining in the United Kingdom

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SYNOPSIS

In recent years a large proportion of capital investment within British Coal has been concentrated on maintaining and extending coal production within the concealed coalfields of Yorkshire and East Midlands. Development of these mines down to depths in excess of 1000 m has necessitated the construction of surface drifts and shafts through Permo Triassic and Coal Measure aquifers from surface down to 750 m.

The following paper briefly reviews the methods of downhole insitu testing, data analysis and inflow prediction developed by British Coal and Birmingham University over the last 10 years to quantify the risk of water inflow from typical Permo-Triassic and Coal Measures aquifers within the UK. The paper describes the development of onsite testing procedures and downhole equipment in responce to the problems associated with the occurrence of high permeability aquifers at depth of between 300– 800 m.

A considerable amount of work has been undertaken at the University of Birmingham to improve not only the techniques of test interpretation but to develop a series of numerical models that are capable of predicting steady and non-steady state inflow rates from multi-layered aquifers exhibition high variations in horizontal and vertical permeability. Advances in techniques of inflow simulation are demanding an improvement and a move away from the standard short duration single hole testing techniques used in the past towards a more sophisticated procedures using an adjacent multi-port piezometer and a longer duration outflow tests.

DEVELOPMENT OF INSITU TESTING TECHNIQUES

The Selby Project 1970–1980

Insitu downhole testing in the 1970's for the Selby coalfield at depths from 300 m to 700 m has been described in detail by Daw (1984) and Fotheringham and Black (1983). All test work was conducted in mud filled holes utilising oilfield tools and testing techniques. Experience in downhole test procedures and interpretation was generally held within the testing company and tests were conducted by engineers unaccustomed to the requirements of a major shaft sinking operation. The downhole test assemblies were not resetable and used compression packer systems that required the full compression of the downhole test string on the bottom of the hole to expand the solid rubber elements. In all cases, downhole test procedures were used according to Horner (1951) and followed a classical Drill Stem Test (DST) approach. Instrumentation within the system was simple, using downhole clockwork Amerada gauges and a surface bubble hose to quantify flow rate. In aquifers of moderate to high strength, exhibiting low to intermediate permeability test procedures worked very well and reliable hydrological parameters were acquired. Unfortunately, severe limitations with this system were experienced when testing weakly cemented aquifers whose caliper and wall strength was irregular and soft eg at North Selby shaft, up to nine attempts were made to test the Ackworth Rock sandstone at between 500 m and 600 m depth. The selection of test horizons therefore was controlled strictly by the caliper and strength of the borehole walls rather than the characteristics of the aquifer itself. Due to the long lead time from test work to shaft sinking, the consequences and lessons to be learnt from inadequate test work were not fully appreciated until the early 1980's when the North Selby shafts intersected the Coal Measure sandstones.

In the early 1980's downhole test work conducted in Warwickshire for the first time used braided steel reinforced inflatable rubber packers that were inflated by means of a rotating downhole pump system. This system, manufactured by Lynes Petrotech very quickly proved itself in poor hole conditions and for the first time allowed test work to be conducted in relatively low strength aquifers. This system was used to investigate the hydrological problems at Asfordby. (Lloyd, Jeffery 1983).

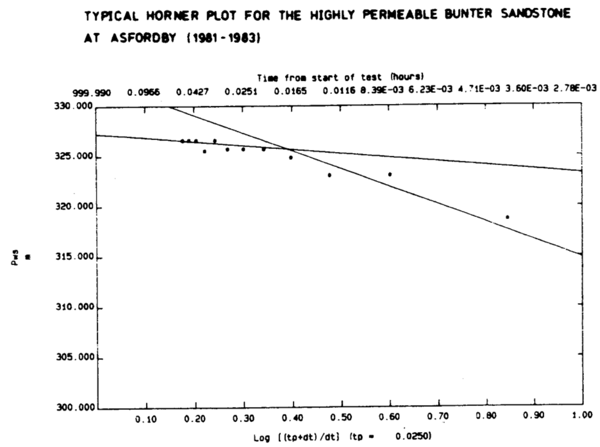

FIG.1

Hydrological Test Work at Asfordby (1981–1983)

One of the principal requirements of the site investigation at Asfordby was to investigate the inflow characteristics of the Permo-Triassic aquifers at between 330 m and 400 m below surface. Early DST tests from an adjacent borehole prior to the shaft site investigations indicated a very high contrast in permeability and strength. Permeability ranged from 10^{-10} to 10^{-5} m/sec and strength varied from 4 to 40 MPa. Due to the limited thickness of the aquifer it was essential that the downhole test work should not only quantify the variation in permeability but that the subsequent modelling work should investigate the inflow potential of the full aquifer system as the shaft approached and passed through individual water-bearing layers.

The test work in both shaft centre boreholes confirmed the reliability of the downhole inflatable packer system. Excellent data for low to intermediate permeability layers was obtained in ground exhibiting irregular borehole geometry. However, problems were encountered when testing the highly permeable Sherwood Sandstone. Within the Bunter Sandstone, early flow rates through the down-hole assembly were very high, often in excess of 3 1/sec. Due to the rather complicated configuration within the downhole assembly, the fluid pathways within the tool severly choked back the flow. With flow periods lasting only several minutes and pressure sampling rates set at 1–2 min intervals, the quality of the test data below the hydraulic shut-in tool was not good. A typical Horner Plot from the Bunter Sandstone illustrating this poor quality is given in Figure 1. To alleviate this problem of poor instrumentation and severe choking a Conductor Wireline Gauge (CWL) was lowered into the drill-string to a position immediately above the hydraulic shut-in tool after the packers were set. The application of a live transducer that could sample head from which the flow rate can be calculated at intervals of less than 10 seconds significantly enhanced the flexibility of the testing system. By eliminating the 'Shut-In' phase of the DST the aquifer was allowed to flow to its final piezometric level. Permeability data could then be acquired from the modelling of this inflow curve (Lloyd & Jeffery (op.cit.)) using a finite difference radial model. The restriction in the early time flow was accommodated in the model as well loss. During the Asfordby site investigation, a series of injection tests were also attempted in an effort to assess the grouting capability of these sandstones. The results of this work were very poor, with the values of permeability generally being up to one order of magnitude lower than that derived from outflow tests. This reduction in permeability was attributed to mud damage incurred while drilling.

From Selby to Asfordby a substantial improvement in downhole testing procedures had evolved; however the work was still very slow (1–2 tests per day) and very expensive (£10,000 per day) and problems were still encountered with the severe choking of the test tool which particularly affected boundary conditions during the modelling of the inflow curve. Over the past five years a total redesign of the downhole test assembly has been undertaken.

Present downhole test equipment

The downhole test assembly in use with British Coal at present is manufactured by Tam International and Exal Wireline Services. A diagramatic section through the downhole assembly is shown in Fig 2. The system utilises up to three braided steel reinforced rubber inflatable packers, that can offer two different packer straddles per test assembly. All of the packers are inflated externally using either water or nitrogen. The system offers no restriction to flow and has an equivalent flow area of 0. 0026 m^2.

Instrumentation is via two live CWL quartz transducers that are strapped to the outside of the test assembly and are tapped into the assembly above and below the valve via two flow ports. The sampling rates of the gauges are less than 3 secs and all downhole pressure transient data is accessed and stored directly onto a surface computer. The downhole assembly is lowered into the borehole on a 73 mm EUE testing string that allows fluid to be evacuated from the test assembly via swabbing. The

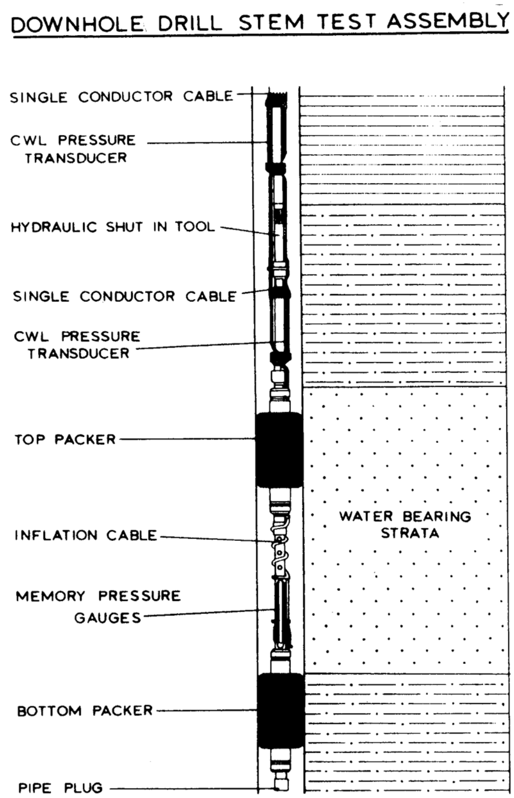

FIG.2.

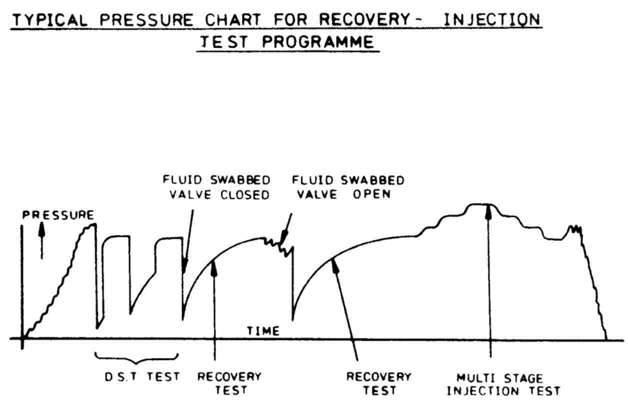

FIG.3.

use of downhole swabbing has allowed the test assembly to be fully resettable and is used to clean up and sample a contaminated test formation. A typical test procedure for a highly permeable aquifer is shown in Fig 3. From Fig 3 it is seen that a multiple test sequence is now available that utilises standard DST testing techniques as well as the inflow test as described in the previous section. If a test formation is thought to be damaged by mud invasion the use of swabbingenables repeat tests to be conducted to calibrate the degree of clean up. An example of a Horner Plot from the DST phase of Fig. 3 using the present equipment is given in Fig. 4

DST DATA INTERPRETATION

The data obtained from the drill stem testing are time variant pressures: no directly measured flow data are obtained. As pressure and flow are required for permeability interpretation, the flow is calculated from the pressure rise within the drill pipe during the DST work.

Equations describing radial flow to an open borehole section were originally derived by Theis (1935). These were simplified by Jacob (1940) to assess a straight-line interpretation of data within certain governing conditions. Horner (1951) adapted the Jacob method for DST and the Horner method is now universally used in the oil industry to analyse data obtained

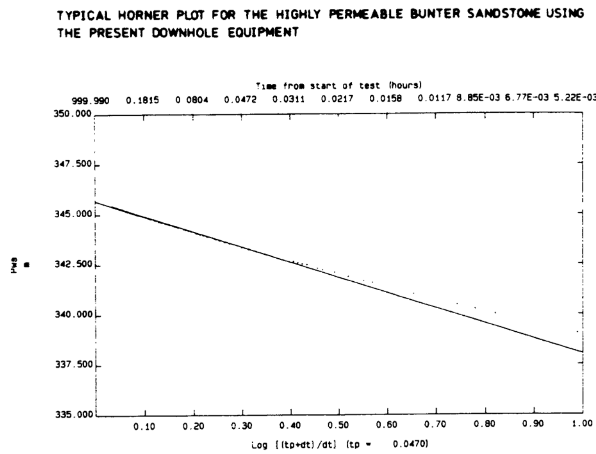

FIG. 4.

from the shut-in parts of the test (Fig 3). For a constant rate build up analysis the flow is approximated as an average value from the proceeding flow segment of the test.

For semi-logarithmic straight line interpretations such as Horner, British Coal use a commercially available package called PANOIL which is operated on site with a micro-computer coupled to the data output facilities. This system also allows for test sections exhibiting non-linear flow and the use of variable rate analysis utilising the technique of superposition. From these interpretations, the degree of damage in a test-section (skin effect) can be estimated in addition to the permeability. The data plot can also be used to estimate formation pressure (static groundwater head). The on-site interpretation provides invaluable understanding about the progress of testing and allows immediate decisions to be taken if changes in the test schedule are deemed necessary.

Permeability derived from semi-logarithmic straight line fits can only be considered as approximations, because the methodology is an approximation of the definitive radial flow equations of Theis, and the method does not account for time-variant flow. Nevertheless, the results are generally considered acceptable within the context of DST work. An example of an interpretation is shown in Fig. 4. Unfortunately due to the effects of boundary conditions the estimation of formation pressure can sometimes be erroneous in shallow groundwater conditions such as those studied by British Coal.

In the oil industry the flow segment data from a DST is analysed typically using various nets of classical responce curves (Earlougher, 1977). However, for British Coal radial flow modifying techniques have been developed for the interpretation of this phase of the testing. The model allows for time-variant flows and well-loss (skin effects) but depends upon accurate formation pressure data: therefore when feasible the flow tests are run until stable head conditions are obtained. This is frequently carried out after swabbing to ensure clean formation conditions.

The equation used for the radial flow model is:

$$\frac{1}{T}\frac{dh}{dr} + \frac{d^2h}{dr^2} = \frac{S}{T}\frac{dh}{dt} \tag{1}$$

where

h is groundwater head (pressure)
r is radial distance
S is the coefficient of storage
T is transmissivity (permeability× section thickness)
t is time

The modelling of the flow segment pressure data is carried out by trial and error, varying the pertinent parameters. The model is sensitive to well-loss and permeability in addition to static head. Boundaries can be applied to the model but are only indicative in that they are clearly radial.

The model is considered to be reliable for low to medium permability interpretation but is not fully developed to handle high permeability conditions. Under these latter conditions rapid, high flow rates occur in the early part of a flow test, which it is believed are non-laminar. As the model relates to laminar flow only, it does not provide satisfactory results and is therefore being modified. The results from radial flow model interpretations are given in Fig. 5.

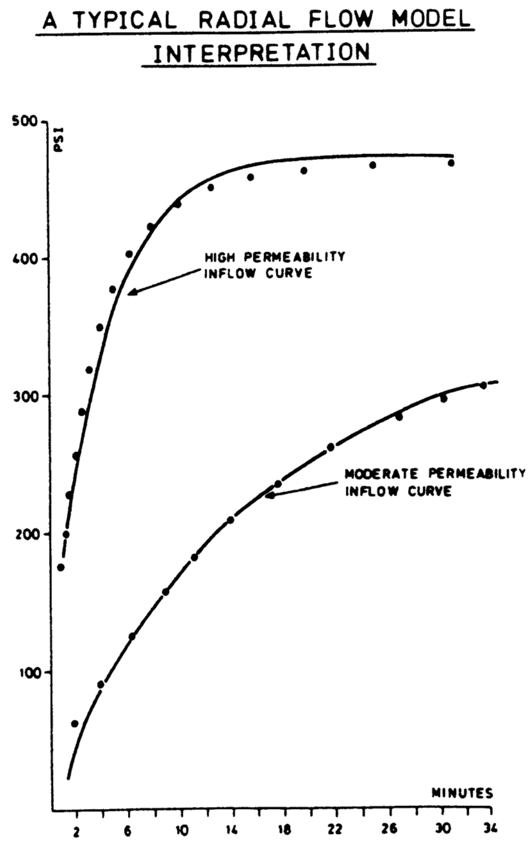

FIG.5.

INFLOW ASSESSMENTS

For shaft inflow assessments, the controlling data are permeability distribution and groundwater head distribution. Heads are obtained directly from the stabilized DST flow segments or indirectly from semi-logarithmic interpretations. Permeabilities obtained from the DST work are very often only from specific sections that are geologically selected. The permeabilities therefore, have to be judged in relation to core permeabilities obtained from the tested borehole and subjectively distributed through the shaft section. In this exercise careful account is taken of the geological, geotechnical and geophysical log data.

Once the necessary distributions have been determined, as for example in Fig 6, the inflows are assessed using radial (r-z) flow modelling techniques similar to those used in the DST inflow interpretation.

Initially British Coal used resistance analogue techniques to represent steady-state flows as noted above (Lloyd et al., 1983). These proved adequate and showed some interesting results notably in respect to inflows at the base of a section of shaft during contruction (Fig.7).

However, the method is cumbersome and has been superceeded by finite difference mathematical methods. The equation used is:

$$\frac{d}{dr}\left(K_r \frac{dh}{dr}\right) + \frac{Kr}{r} \cdot \frac{dh}{dr} + \frac{d}{dz}\left(K_z \frac{dh}{dz}\right) = 0 \qquad (2)$$

where h(r,z) is the groundwater head distribution and K_r and K_z are permeabilities in the radial and vertical direction.

Equation (2) is for steady state flow and may be generally applicable for shaft sinking operations in that the rate of shaft construction does not permit free flow to an excavation over the full depth of a stage under non-steady state flow conditions. The flow domain is shown in Fig.8.

To provide a more comprehensive model however, a non-steady strata version has been developed (Edwards, 1985; Lloyd and Edwards, 1988) based upon the following equation:

$$\frac{d}{dr}\left(K_r \frac{dh}{dr}\right) + \frac{K_r}{r}\frac{dh}{dr} + \frac{d}{dz}\left(K_z \frac{dh}{dz}\right) = S(r,z)\frac{dh}{dt} \qquad (3)$$

and relating to the same flow domain as in Fig.8.

In order to verify the model, it was tested against shaft inflow data from North Selby No 2 shaft in Yorkshire. Unfortunately it is not easy to obtain comprehensive data for verification. Nevertheless the example used has provided very encouraging results. The base hydrogeological data are given in Fig.9 and show the paucity of permeability data. For the model a fixed head was imposed at 500m (Edwards, 1985) with the head 3m above ground

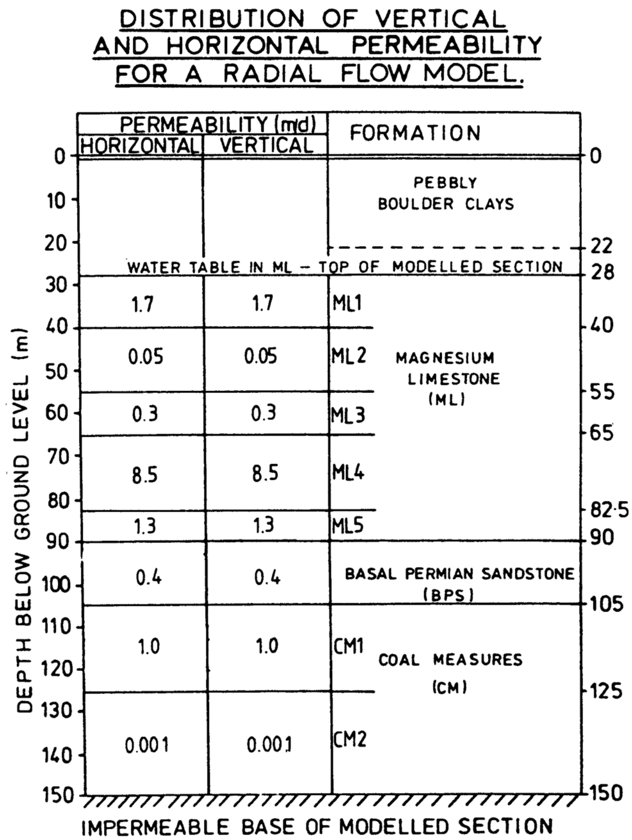

FIG. 6.

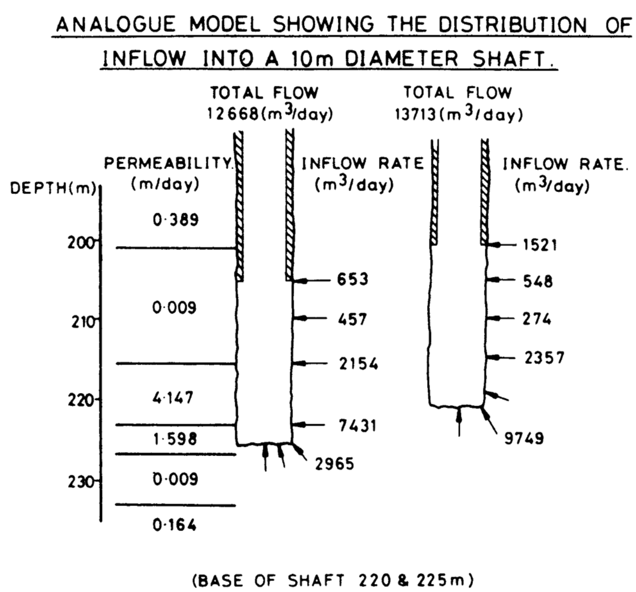

FIG. 7.

Shaft sinking records are available for the depth of the base of the shaft, depth of the base of the shaft lining and the recorded shaft inflow at weekly intervals. These data are shown in Fig.10. For example, on day 15 of construction through the shafton sandstone on conditions the shaft base was at a depth of 608.6 m and the shaft lining to a depth of 599.1 m. The length of shaft open on day 15 therefore was 9.5 m as the inflow was recorded as 609 m^3/d.

Details of the construction process between the weekly recorded times are not known, and it has been assumed that sinking and lining proceeded at a steady rate although two scenarios were examined:

Simulation A—one week time-step

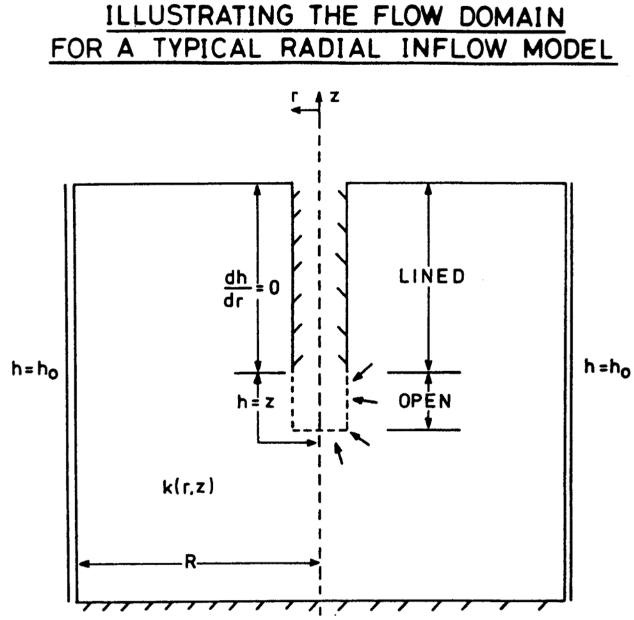

FIG. 8.

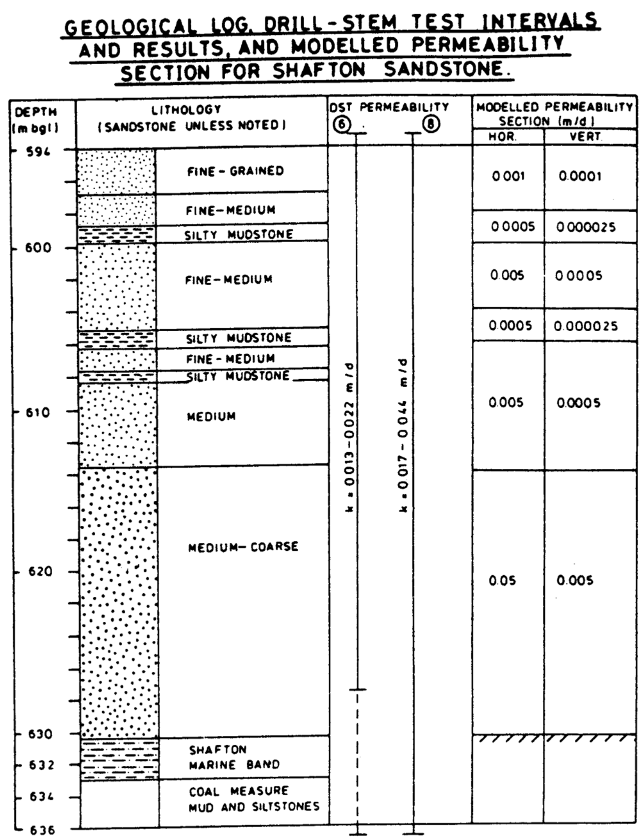

FIG. 9.

Simulation B—construction time-steps of

up to 2 days

The results of the simulations are shown in Fig.11. As specific storage is an important parameter in the non-steady state equation values were varied as part of a sensitivity test.

The modelled inflows are of the correct order of magnitude at all depths with s=10^6/m. More accurate model inflow, it is believed, could have been obtained if the permeability of the uppermost part of the shaft in sandstone was greater than 9.0E^{-4}

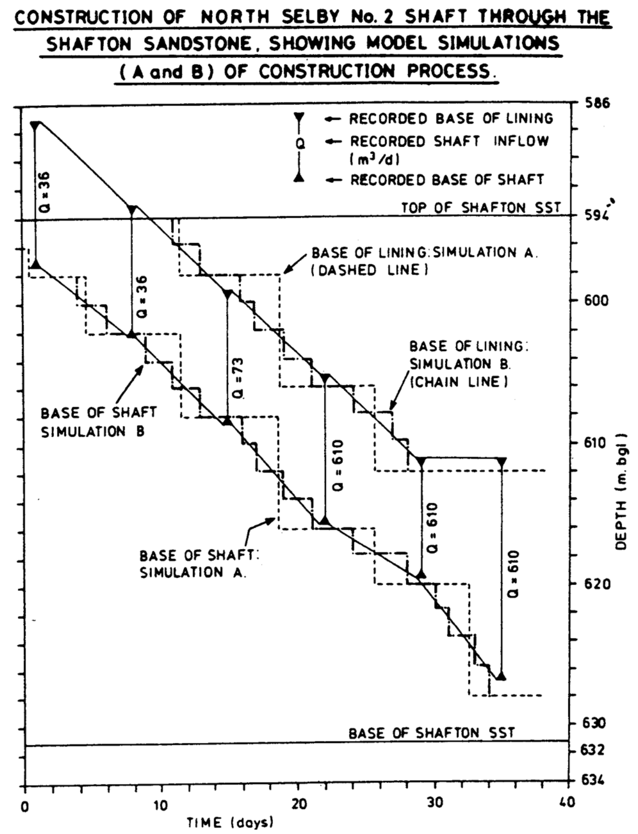

FIG.10.

m/day (Fig.9) and the permeability of the ground below 614m was slightly greater than $5.0E^{-1}$ m/day. This indicates the difficulties that exist in estimating a model permeability section from averaged field data and geological logs and reinforces the need for the comprehensive testing and data interpretation procedures being adopted by British Coal which are described above.

CONCLUSIONS

Although the test methods and interpretations described above are currently used by British Coal in deep shaft investigations, shallow shaft and drift studies have indicated the importance of testing for permeability using cross-hole drawdown interfaces. For this type of work multi-port piezometers are installed in an observation well and flows are created in an adjacent well. The head responses can be interpreted to assess the importance of layered vertical and horizontal permeability. This technique is currently being developed for use at depths in excess of 400 m.

The amount and sophistication of any hydrogeological site investigation programme for a shaft sinking operation will always be determined by the degree of risk inherent in terms of the inflow prediction and the method of ground treatment. However the use of sophisticated numerical models demand the acquisition of a representative set of aquifer characteristics that cannot always be derived from single hole drill stem test methods. Developments within British Coal and Birmingham University are continuing with reference to multi-port interference testing and the use of non-laminar inflow models.

This paper has reviewed progress in the field of insitu-testing and inflow prediction and has illustrated the importance of developments within each field progressing in parallel and not in isolation.

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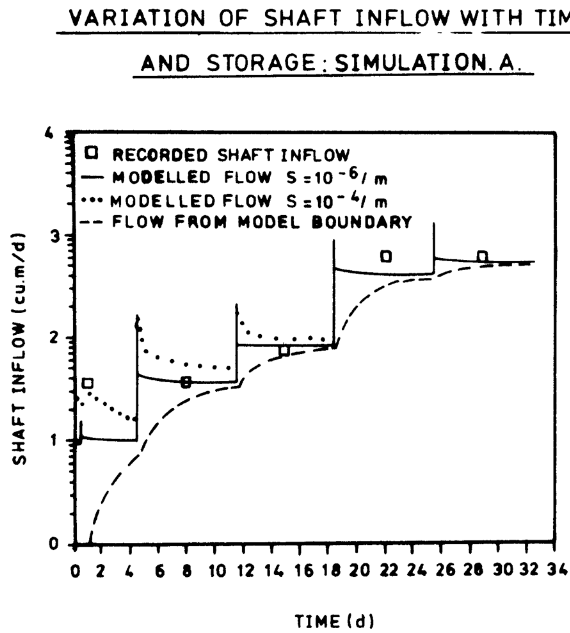

FIG11a

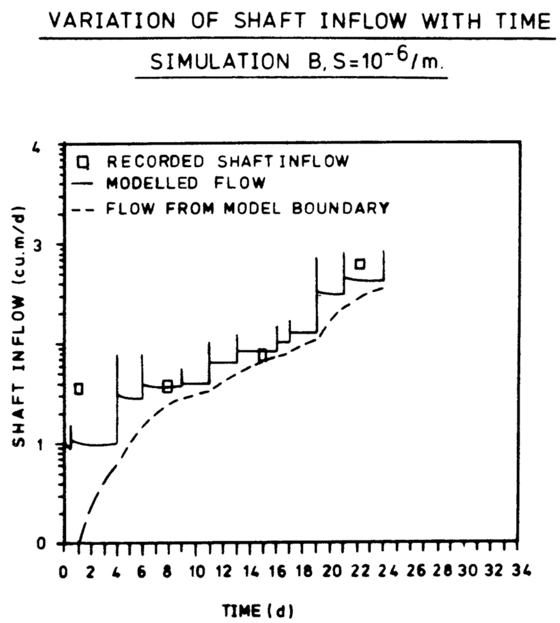

FIG 11b

References

1. Edwards, M.G., 1985. Two-dimensional numerical modelling of inflows to a drift through multilayered permeability ground. Sec. Int. Mine Water Cong., Granada, Spain, Sept. 1985, Proc. Vol. 1, 455–466.
2. Edwards, M.G., 1987. Mathematical modelling of groundwater inflows to shafts and drifts under construction. Ph.D. thesis, University of Birmingham, U.K. Slop.
3. Horner, D.R., 1951. Pressure build-up in wells. Proc. Third World Petroleum Cong., The Hague, Section II, 503–523.
4. Jacob, C.E., 1940. On the flow of water in an elastic arterian aquifer. Am. Geophys. Union Trans. 72, II, 574–685.
5. Lloyd, J.W. and Jeffery, R.I., 1983. Deep aquifer testing methods and data interpretation. Z. dt. geol. Ges., 134, 871–884.
6. Lloyd, J.W., Rushton, K.R. and Jones, P.A., 1983. An assessment of groundwater inflows into a proposed shaft and drift in the Warwickshire Coalfield using packer test data in an electrical analogue model. Int. J. Mine Water, 2 (4), 1–18.
7. Lloyd, J.W. and Edwards, M.B. 1988. Estimation of groundwater inflow to an underground mining operation. Int. SL. Mine Water, 7, (1), 25–47.

8. Theis, C.V. 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using groundwater storage. Am. Geophys. Union Trans. 16, 519–524.

London Water Ring Main tunnel shafts as pumping stations

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ABSTRACT

To meet London's Water Supply needs into the 21st Century Thames Water have developed an integrated strategy for future water supply in the capital. At the core of this strategy is the construction of over 60km of deep tunnel (the London Water Ring Main) to convey water to the capital's major demand centres. The ring main will radically alter the way in which water is supplied to London

INTRODUCTION

Thames Water supplies over 5 million people in the London area with an average of 2300 Ml of water per day. The quantity of water put into supply has been increasing at a rate of approximately 1 per cent per annum since the turn of the century.

Over 70 per cent of London's water is derived from the River Thames and is treated at six major stations situated upstream of Teddington Weir. These supplies are supplemented from the River Lee, in the east, and from underground aquifers in the north east and south east of the Metropolitan area. All these latter sources have been fully exploited and future growth must be met from the River Thames, or from initiatives like artificial recharge of the chalk aquifer.

The majority of London's treatment works are situated on the fringes of the capital and supplies are maintained to the major demand centres by pumping through a network of large diameter trunk mains radiating from the treatment centres across London.

This supply pattern has not changed for 120 years. The trunk distribution system is totally dependent upon pumping and is currently operating at or beyond its capacity.

To meet future needs, Thames developed an integrated strategy for water treatment trunk distribution. At the core of this strategy is the construction of a deep water main in tunnel, encircling London, to supply treated water to the major demand centres where it will be pumped direct into supply.

LONDON'S PRESENT WATER SUPPLY SYSTEM

The pattern for London's water supply system today was laid down in the last century. Originally Londoner's abstracted water from sources within the city, including the tidal section of the Thames. As London developed, so the river water in the tidal reaches became increasingly polluted and hence undrinkable. The shallow springs and wells within London suffered a similar fate. In 1852 the Metropolitan Water Act was passed and with it two important provisions; that all water abstraction from the Thames should be from non-tidal sections and that all water should be effectively filtered. As a consequence, the Companies based on the Thames were required to move their works further to the west above the tidal sections and they were obliged to invest in filters and the transfer of treated water eastward back into London.

Major treatment stations were established in the Thames Valley upstream of Teddington Weir at Surbiton and Hampton. Both of these are still in operation today, supplying some 29 per cent of London's treated water. Further development has taken place and today there are works at Walton, Kempton Park and Ashford Common.

In parallel with developments in the Thames Valley, further works were established to the east of London in an area which is today known as the Lee Valley supply area. In 1642 a conduit, the New River, was constructed to convey River Lee water from Hertford to Islington. By the turn of the century the New River was supplying water to works at Hornsey and Stoke Newington. A further treatment works at Lee Bridge abstracted water direct from the River Lee, at Hackney. Today the New River still supplies Hornsey and Stoke Newington works, although the works at Lee Bridge has been replaced by one at Coppermills, which was commissioned in 1971.

Supplies to London are based on a distribution system comprising over 9.000 miles of water and mains and 60 separate supply zones. These zones are pressure regulated and have their origins in the old Water Company Acts which set down supply criteria in terms of available head. The natural topography of London has also played an important part in their formation. Zones to the north of London are in the main served from dedicated service reservoirs, whereas south London zones are categorised into three pressure levels; low, high and very high. The pressure difference between these is as much as 70 m.

Because of the way in which the trunkdistribution system has been laid down, each treatment works supplies fairly distinct areas of London. Mains capacity far in excess of the original needs was originally provided, and over the years some flexibility has been introduced to the system through cross-connections in the trunk system and provision of facilities for infusions between zones.

CONCEPT AND STRATEGY

Thames decided some years ago that there were two broad options for the future.

1. To continue with a programme of replacement of the existing system through the provision of additional surface trunk mains and remodelling of existing treatment works, or
2. To construct a gravity fed ring main (Fig. 1) linking modernised uprated works with the major demand centres.

Thames decided that the latter option had a number of advantages associated with the Ring Main concept namely potential financial savings, energy savings, reduction of leakage and fractures in trunk mains, and the avoidance of cost and disruption associated with laying mains.

Development plans for London's water treatment and trunk distribution strategy based on the ring main were endorsed by the Authority in April 1985.

RING MAIN

The ring main will comprise nearly 80km of 2.5 m diameter tunnel, including the existing 19 km of southern tunnel main, which it is proposed should be integrated with the ring main system. The proposed transfer capacity of the ring configuration is 1300M1/d of fully treated water representing over 50 per cent of London's present daily demand. The remainder will be supplied through the existing trunk mains.

By virtue of the ring system any one of the outlet shafts can be fed from either of two directions. Similarly, should one treatment works be incapable of feeding the network, then another could supply. The main driving force in the movement of water will be gravity, the system being pressurised under the head of the treatment works, which are sited approximately 30m above the proposed tunnel axis level. At the outlet shafts, water will be lifted to the surface and pumped directly into the local distribution system.

ENGINEERING CONSIDERATIONS

Tunnel axis levels have been determined so as to ensure that the minimum pump suction head is achieved under all flow conditions. However, choice of construction method has played a important part in determining the actual depth at which the tunnel will be driven. The philosophy of the ring main is dependent on the tunnel being driven deep into the London clay strata to take advantage of modern soft ground tunnelling techniques using pre-cast concrete expanded linings which depend on the ground overburden for their stability.

Considerable expertise has been gained by Thames Water in the use of the 100 inch wedge block tunnelling system, which was originally developed within the former Metropolitan Water Board. However the majority of tunnels constructed using this method have been for raw water transfer between storage reservoirs. Principal amongst these is the Thames Lee Tunnel, which links the Thames and Lee Valley storage reservoir systems. Only one treated water tunnel, the southern tunnel main, has been constructed so far using this system.

The original concept and costings were based on use of the wedge block system. However, ground investigation soon revealed that this would not be possible in all cases, and that some 10km would have to be driven even deeper into the Woolwich and Reading beds to avoid existing utility tunnels.

Use of the traditional bolted segment with in situ concrete linings for these sections would prove very expensive and further alternatives were explored. The most promising of these was a modification of the smooth bore type of concrete lining originally developed for sewer tunnels in ground other than wedge block clays. It was necessary to modify the design of the smooth bore segment to accommodate rubber gaskets required to eliminate the possibility of leakage and also to withstand the considerable ring tension loads developed from the internal pressures. Designs were eventually evolved which theoretically ould seal and have structural integrity without external ground support, as this could not be guaranteed in all situations.

Extensive testing work was necessary and this was carried out with the segment manufacturers as the principal supplier. Tests were undertaken using both normal and hydrophilic rubber gaskets, with a measure of success up to 2 bars internal pressure. However, it soon became apparent that great care was needed to achieve successful sealing whatever system was used. The results of these tests were made available to tenderers, but they were also encouraged to submit alternatives either based on the results of these tests or on their own design.

PUMPOUT SHAFTS

As far as possible a common design layout has been accepted for shafts, shaft fittings and pump layouts Fig. 2 although as can be expected with up to twelve sites local variations have occurred. The basic shaft is of double diameter construction 11.9 m diameter down to motor floor level and 10.3 m diameter below.

Soils investigations indicated that due to the presence of overconsolidated clays high compressive forces could be expected in the long term. The basic precast shaft lining has therefore been designed to accept a ratio of horizontal to overburden stresses of 1.5 to 1. In addition load cells are installed into the clay to monitor any changes in horizontal pressures. Should these pressures exhibit a tendency to increase with time, room has been allowed for the strengthening of shafts by means of in situ concrete linings to the critical sections. All precast linings have been installed with hydrophilic rubber gaskets to control groundwater ingress.

Shafts sunk to date have been constructed by underpinning, although where the overlying stratum comprises water-bearing gravels, caisson methods have been used down to the underlying clays. Connection from the tunnel into the shafts is achieved with steel tapers which extend into the tunnel approximately 4 m. The tunnel diameter through the shaft is 1.8 m connected by 1.8 m diameter valves to a 1.8 m diameter manifold in the form of a cross. The manifold is provided with connections for up to six pumps. In most shafts all six will be installed.

Because only one valve is employed on each side of the manifold, the design of these valves (fig. 3) incorporates certain features to provide a high degree of integrity when shut. This is necessary to give confidence and absolute security to those operators whose job it will be to enter the tunnel for inspection, even though manual inspections are not anticipated on a frequent basis.

The valves are designed to seal on the upstream face, any pressure forcing the downstream face tighter against its seating being a bonus. When shut, drain valves will allow the body cavity to be emptied and should leakage occur past the upstream seal, this can be identified and measured. Should it prove that seal wear has occurred, then the gate and seals may be removed and the seals renewed without interfering with the operation of the tunnel. This is achieved by winding the gate fully up into the valve bonnet. A stop log is then inserted below the gate to be removed. By this means and also with a routine six monthly operation, the valves should give a satisfactory life expectance.

In addition to the built-in maintenance facility, valve body integrity will be checked at intervals. This is achieved by undertaking a thorough non-destructive inspection of critical welds during manufactuer and immediately after installation. By this means a fingerprint of each valve will be established. Subsequent tests may then be compared with the original fingerprint and any variation noted. It is expected by this means that any defects, which may affect the valve's integrity, can be identified before they cause problems and early enough for controlled and planned maintenance procedures to overcome them. Similar procedures are planned for other critical items.

One of the most important areas in the design is the sizing and selection of pump units. Sizing requirements have been derived from hydraulic network analyses of individual zones and from the GINAS Simulator.

Initial analysis to confirm the original justification of the scheme was undertaken using WRc 'WATNET' program. However, 'WATNET' did not go far enough in analysing the interactions between component parts of the system. In order to determine the detailed modes of operation for individual supply zones a more interactive model was required. In 1986 Leicester Polytechnic were commissioned to produce a dynamic model of the main and primary trunk distribution systems, including reservoirs and existing pumps utilising the 'GINAS' suite of programs. These programs together with detailed network analyses of existing distribution zones will enable Thames to develop a control and coordination system which will ensure the efficient use of the ring and existing systems to guarantee security and reliability of supply, and maximise the projected cost benefits.

Because of the base load mode of operation proposed for the ring main, the majority of pump units can be of the fixed-speed type. In the few locations where a variable input will be required, then it is proposed to install variable-speed units which will have the capabilities to supply at least two zones.

Initially, it was proposed to install submersible type pump units, although now it is intended that more conventional direct-drive units should be used as they have been demonstrated to be more cost efficient. Motors ill be installed within the shafts on a landing just below the shaft top (Fig. 2). The pump impellers will be sited just above the manifold level, connected to the motors via drive shafts inside the rising mains. However, as as in the case with tunnel linings, alternatives are encouraged provided that they can show equivalent cost benefits.

In addition to the main supply pumps, drain down and sump pumps will also be installed. The tunnel drain down pumps will have the capability to pump into the supply mains should this facility be required for further standby.

The power requirement for the pumping stations are considerable and discussions have been undertaken with the Electricity Boards for the provision of suitable power supplies to the various shafts. Should, for any reason, a power failure occur during operation of the system, provision has been made for a level of standby by duplication of the main electrical feeders. Previous policy within Thames Water would have dictated installation of generators to cover up to 75 per cent of the pumping duty. A considerable saving in capital will be realised with the introduction of dual feeders, a decision which has been based on a comprehensive risk analysis.

One of the potential problems which required resolution was that of surge caused by sudden valve closure or pump shutdown. Valve closure is governed by actuation mode and will take minutes rather than seconds. Analysis has shown that valve closure will not cause critical surge. Pump shutdown due to power failure cannot be ruled out and following discussions with London Electricity Board a worst case was agreed whereby three adjacent shafts could shutdown together, this case was analysed and surge pressure established.

Due to the method of construction wedge block tunnels can only withstand limited internal pressures.

Cooley in his paper on wedge block tunnels recommends that surge head should not exceed 1.5 times overburden cover. This has been further refined, to give an additional factor of safety of London Water Ring Main by the use of surge towers within pumpout shafts.

The surge wave is damped by friction in the relatively small diameter of the tower and any overflow, should the wave reach ground level, is collected by an overflow chamber and discharged to waste.

Shaft locations have wherever possible been situated on sites already owned by the Authority, there is however one notable exception at Park Lane. This shaft is located on a traffic island in the middle of Park Lane just outside the Hilton Hotel and adjacent to Byron's statue. Work on this shaft commenced in June 1988 and it should be operational by the end of 1990. Even though acquisition of this site caused problems, work on our own sites has raised greater difficulties. Especially in respect of disturbance to residents, lack of space, and Local Authority planning requirements. At two sites considerable problems arose due to local residents complaints, even before construction, of loss of amenity, disturbance and reduction in property values. Fortunately none of these factors has in fact become critical and work proceeded with few public relations problems.

Implementation of the ring main will considerably advance water supply technology within Thames and in terms of operational effectiveness and efficiency place London's water supply system on the map.

ACKNOWLEDGEMENTS

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The author would like to thank the following people who have made contributions in the preparation of this paper, Mr I H Bensted, Planning Manager, Operations Directorate; Mr M A Keane, Mr N Britton and Mr R Harrison, Planning, Operations Directorate.

In addition, the efforts of the employees of former undertakings, which now comprise Thames Water, who originally developed the concept of the ring main and who developed the techniques by which this has become economically possible, are gratefully acknowledged.

REFERENCES

The following references give further information on the development of the ring main:

1. Cooley P Wedge block tunnels in water supply. J Inst Water Eng Sci, 1982, 36, 9–26.
2. Thames Water, Internal Report, Alternative strategies for London's water supply system, Water Planning Committee, 20th October 1984.
3. Thames Water, Internal Report, Future water treatment in London, Report of Working Group, January 1984.
4. Keane M A, Harrison R and Britton N London's water supply in the 21st century: computer modelling aspects, International Conference Computer Applications for Water Supply and Distribution, 1987.
5. Thames Water, Internal Report, London's water supply in the 21st century: a strategy for water treatment and trunk distribution, April 1985.
6. Keane M A and Kerslake J C The London Water Ring Main an optimal water supply system. Journal Inst Water and Environmental Management, June 1988.

Groundwater control during shaft sinking

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ABSTRACT

Mine-water inflows not only threaten the miners and mining economics, but also the cost and quantity of water resources in the region. A grouting technique is now widely used in the USSR to eliminate or reduce the flow of ground water into existing or proposed mine shafts. This resulted from the development in the early 70s of a grouting method based on the integration and outgrowth of groundwater hydrology, grout chemistry and rheology, rock and soil mechanics, drilling and mining engineering.The paper gives a general review of technical philosophy and approaches with regard to grouting technology introduced and being practiced by the STG Association, the largest specialist group in the Soviet Union. A number of case histories is mentioned, together with the description of specifice aspects of shaft grouting process and design.

INTRODUCTION

Over recent years a specialized grouting organization 'STG' from the USSR has developed and refined the so-called 'Integrated Grouting Method' which is being used now for shaft sinking programmes in fissured and karst rock environment. This technique has been successfully implemented in 106 various mine shaft projects in the USSR, Hungary, Czechoslovakia, Bulgaria, Romania and has included projects related to shaft sinking up to 1600 m deep, drilled shafts with a finished diameter of up to 4.5 m and 1050 m deep, shaft repair operations and shaft reopening programmes.

GENERAL DESCRIPTION OF TECHNOLOGY

The technology developed within STG involves three major stages:

- investigation and data acquisition
- design and costing
- implementation and testing

Investigation is carried out with the specific objective of providing criteria for design and costing of the grouting programme. This will entail the determination of the grout curtain size and interlocking covers spacing, grout volume to be implaced, pressure of implacement, grout holes pattern and depth.

The process which is followed in investigation includes the drilling of one normal diameter core hole. The hole is geophysically logged on completion and flow metered. This allows permeable zones to be grouped for later grout design and injection. The objectives of in-situ testing for grout design are related to evaluation of rock voidage. hydrostatic head, hydraulic conductivity of the medium, hydraulic anisotropy.

The evaluation of the grouting environment determines the characteristics of the grout formulation required and the nature of the grouting operation considered appropriate. The grout formulation must be developed then to have:

- rheological properties compatible with the fractures, fissures and water conduits
- physical properties which will permit grout propagation through the environment under the pressures and at the temperatures which apply
- chemical properties both during implacement and long term which will be compatible with the environment and which will maintain gel strength
- set-up strengths and plasticity adequate to cope with post-grouting strata movements

Clays from the local project environment are assessed in this regard and are subjected to detailed testing in order to estimate their suitability for formulation. This includes assessment of their mineralogy and chemistry, hydration and dispersion characteristics, filtration characteristics, economics of excavation, transport and chemical conversion to establish the properties desirable.

Commonly used stabilized clay grouts consist of a chemically modified clay as the filler mixed with water of a compatile salinity and chemistry. Other reagents include the binding agents (e.g. cement, fly ash, dolomites, etc.) and structure-forming additives. Other chemicals are used to stimulate dispersion, hydration and to achieve flow and set-up properties and times. The water used in mixing is carefully controlled as to chemistry and volume and the chemicals are added to a specific schedule in order to achieve the grout slurry properties required. The specific gravity of the slurries is aimed at between 1.20 and 1.23 t/m^3, but is a property which may be varied from project to project dependent upon the pressures desired during grouting.

In implementing shaft grouting programmes STG prefer to have their mixing facilities at the surface irrespective of whether it is grouting in advance of the development of underground openings or not. While this may involve the additional cost of a conveying borehole to the grouting face, it has many advantages: avoiding disruption of development operations, removal of machinery, high pressure pipelines and fittings away from confined working spaces, acceptability of large mixing and storage equipment, avoidance of the need for personnel trained in underground working environments.

Grout placement is generally undertaken progressively moving from hole to hole, completing one defined zone after another. By flow-meter testing in advance the ungrouted transmissivity of each zone is determined (Tmax) and the client's acceptable residual inflow is converted to the desired end transmissivity (Tmin). By testing each ungrouted hole on completion of grouting in the adjacent hole plots of the declining residual transmissivity are developed. This ensures that over grouting is not undertaken on any zone or overall.

GROUTING OF FISSURED ROCK STRATA AND KARST ENVIRONMENT

Over recent years STG has completed a number of projects associated with grouting of fissured aquifers and karst environment during sinking of shafts up to 1200 m deep both through a series of 4–6 directionally drilled holes and one single hole following a spiral path around the proposed shaft. At present one similar project is in progress in Romania at the Palazu-Mare iron-ore deposit. It is related to the drilled shaft with the total depth of 650 m and final diameter of drilling 3785 mm to acommodate 2960 mm dia. casing. The problem consists in the necessity to drill the shaft through 450-m thick intensively karstified Jurassic water-bearing formation in the 50–500 m shaft section. Permeability of karst strata amounts to 40–50×10 −12 m^2 with some caverns up to 5 m in diameter. Karst voids are partially filled with saturated clay-sand material characterized by quick ground properties.

Shaft drilling in such conditions is complicated by catastrophic lost circulations, collapse of shaft bore walls, the necessity to case separate zones with additional casings, as well as the difficulty of reliably cementing the casings in such an environment.

To provide reliable sealings of karst strata, to exclude lost circulation and ensure effective cementation of casing, a pregrouting programme is being carried out in the shaft site. Grout injection is performed through 4 holes of 112 mm diameter drilled to a depth of 510 m. To achieve maximum combination with shaft drilling, grouting operations have been designed to proceed in two stages.

At the first stage karst strata is treated to a depth of 260 m simultaneously with shaft drill rig mounting. The designed quantity of grout injection at this stage equals 20,000 m^3. At the second stage the strata is treated to a depth of 500 m simultaneously with shaft drilling to a depth of 200 m. The designed volume of grout injection at the second stage is 4000 m^3.

Implementation of the grouting project for the drilled shaft PA-I at the PalazuMare deposit will ensure the simplification of shaft bore casing (Fig. 1) and considerable cost saving.

SEALING OF ABANDONED MINE WORKINGS

The problem of sealing of abandoned mine workings often arises during shaft sinking through mined-out zones or flooded mine levels. A special technique has been developed to prevent inundation of mine workings during implementation of shafting programmes.

This technique can be illustrated by a grouting project designed to control ground water inflows into a drilled shaft at the Kuybishevskaya Mine, Donets Coal Basin. The shaft bore was to traverse disused mine workings at the 270 m level while being drilled to the final depth of 550 m. In accordance with hydrodynamic testing data the opening of worked-out strata was calculated to amount to 0.3– 0.4 m. The proposed shaft had 2600 mm final diameter of drilling to accommodate 2100 mm dia. casing. Four inclined grout injection holes were drilled to a depth of 285 m. On the completion of drilling the grouting holes were geophysically logged and water tested to obtain accurate data for grouting process design.

The designed quantity of grout required to seal the worked-out strata was 11,000 m^3. Actual propagation of grout around the shaft bore was controlled by borehole acoustic television method. On the completion of grouting the shaft bore has been drilled without any complications.

RESIDUAL SEEPAGE CONTROL IN CONCRETE LINED SHAFTS

The pregrouting technique with cement grouts has been used to control ground water inflows during the sinking of the vent shaft at the Voikova Mine, Donets Coal Basin. The programme failed to be a complete success and the shaft has been sunk to the proposed depth with a residual seepage of 21 m^3/hr.

A post-grouting programme has been designed by STG to cut down the shaft inflow to standard regulations (5 m3/hr). The grouting scheme arrangement involved drilling of inclined 105 mm dia. holes through the three waterlogged sections: 24–34 m, 49–60 m and 80–104 m. The depth of grout injection holes was in the range of 15 to 30 m. Grout preparation and pumping equipment was deployed on the ground surface to inject the estabilized clay grout via a high-pressure pipeline. The grouting operations in the shaft were carried out from temporary platforms mounted at levels of 22, 47 and 75 m. The post-grouting programme allowed residual seepage to be reduced to 4.8 m3/hr.

RESIDUAL SEEPAGE CONTROL IN TUBBING LINED SHAFTS

The service shaft at the Melnikova Mine, Donets Coal Basin, traverses a porous sandstone aquifer in the section of 589–649 m. Drilling of 3 pilot holes resulted in a total borehole yield equal to 240 m3/hr at 1.7 MPa pressure head. The maximum discharge of 180 m3/hr was received from one hole. Ground water temperature was 30°C.

To ensure safe shaft sinking through the sandstone strata, freezing by liquid nitrogen was performed through 60 holes 55 m deep. When freezing was completed, shaft sinking proceeded to a depth of 614 m with tubbing lining and backwall grouting by a sand-cement mixture. At a depth of 614 m the shaft inflow increased from 3 m^3/hr up to 60 m^3/hr.

STG has undertaken the project to deal with the critical inrush of high-pressure water into the shaft excavation. Remedial operations included the construction of a reinforced concrete plug in the 610–614-m shaft section. The plug design comprised a drainage layer with a relief pipe. Ground water pressure was 1.7 MPa.

Grout preparation and pumping equipment was deployed on the surface. The back-wall zone of tubbing lining was treated with 85^3m of special clay-cement grout formulated for a low-temperature environment. Backwall grouting was followed by grouting of the drainage layer. As a result of these operations, residual seepage was cut down to 0.5 m^3/hr.

In the course of further shaft sinking the inflow at a depth of 649 m again increased to 28 m^3/hr. The waterlogged section was treated with an extra 144 m of low temperature grout which enabled the shaft inflow to be reduced to 0.4 m3/hr.

SHAFT DEEPENING

The modernization project of the Ukraine Mine, Donets Coal Basin, involved the deepening of skip shaft No. 3 from 440 m to 750 m. The anticipated inflow from this zone into the proposed shaft was 152 m^3/hr.

Three inclined grouting holes 340 m deep each were drilled from the 410-m level mine workings to treat the water-bearing layers. The arrangement of grouting holes allowed grouting operations to be carried out simultaneously with equipping the shaft for sinking. The total quantity of grout injected within a 5-month programme amounted to 4620 m^3. During shaft-deepening activities, residual seepage into the shaft did not exceed the client's specifications.

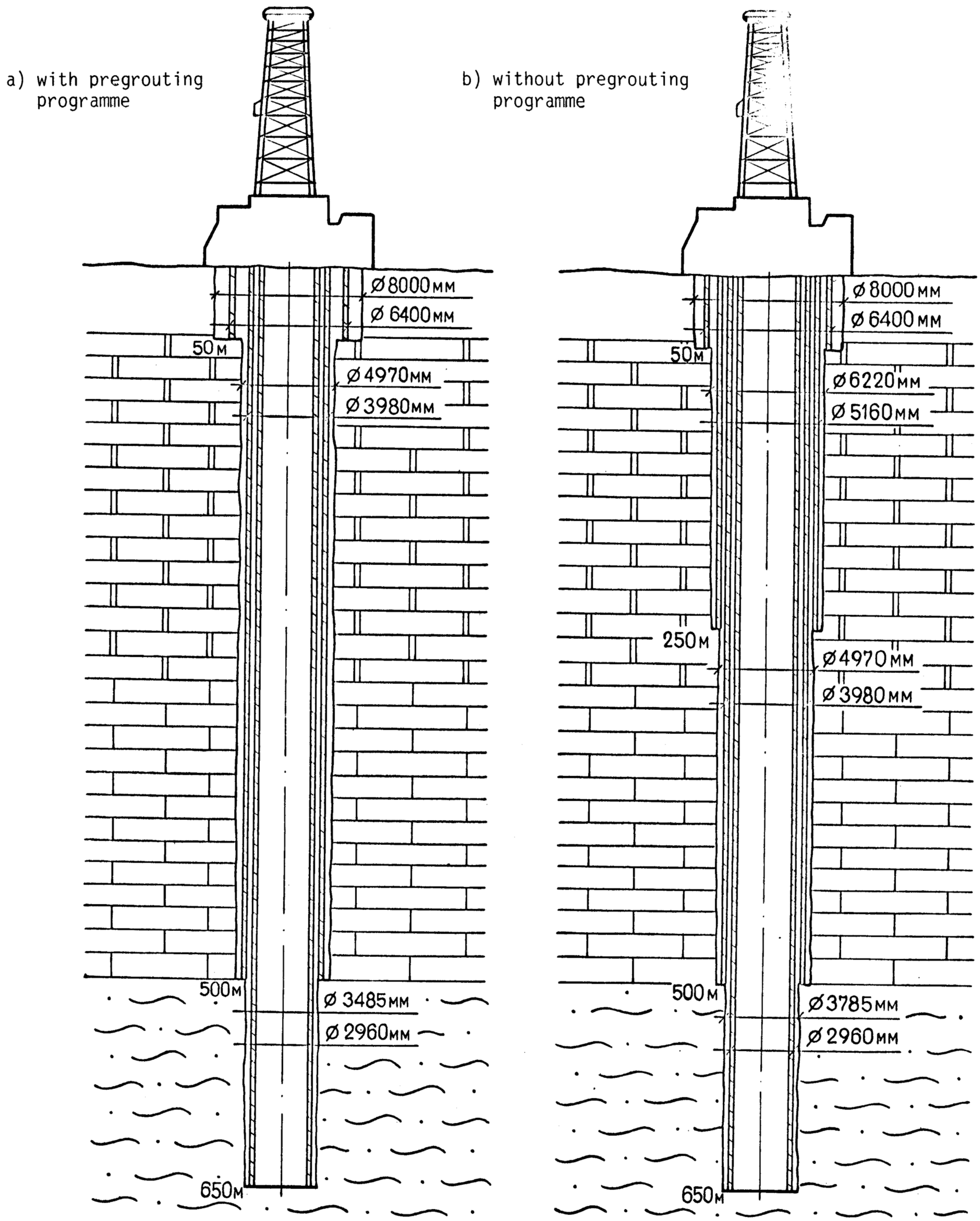

Fig. 1 Drilling and casing arrangement for shaft bore PA-I in Karst environment (Palazu-Mare ore iron deposit, Romania)

Excavating large-diameter boreholes in granite with high-pressure water jetting

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SYNOPSIS

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Experiments pertinent to the Canadian nuclear fuel waste disposal concept are to be carried out "in situ" in granitic rock at Atomic Energy of Canada Limited's Underground Research Laboratory (URL). Two of these experiments require boreholes 1240 mm in diameter and 5 m deep to be excavated in the floors of rooms in the URL. A coring technique using high pressure rotary water jet rock slotting is being developed to excavate these large-diameter boreholes.

The coring rig incorporates a rotary, double water jet nozzle to cut a vertical, circular slot, the outside diameter of which equals the required borehole diameter. To facilitate core breakage and removal, two nonrotary, water-abrasive nozzles are used to cut a horizontal notch in the core. The nozzles, which operate at a water pressure of 135 MPa and a water flow rate of 50 L/min, are mounted on coring sections which rotate in the vertical slot.

The stages of development of the coring rig have involved nozzle testing and optimization, shop tests in large granite blocks and field trials at the URL. This work has shown that a coring production rate of between 190 and 240 mm/h can be achieved, and an 80 mm deep undercut slot can be cut in the core. This depth of undercut slot is sufficient to break the core with expanding cement. During the URL trials, difficulties with cuttings removal at depths greater than about 0.5 m and occasional jamming of the coring sections during vertical slotting led to the development of procedures necessary for successful excavation of the large-diameter boreholes. These procedures include the use of a sump hole to collect the cuttings and staged removal of the core as the hole is advanced.

This paper describes the stages of development and experience gained in the operation of a prototype water jet coring machine to drill large-diameter boreholes in granite.

INTRODUCTION

The Canadian concept for nuclear fuel waste disposal proposes that the waste be emplaced in a vault located at a depth of 500 to 1000 m in stable plutonic rock of the Canadian Shield. The general configuration of the reference vault is illustrated in Figure 1. Excavation requirements include shafts for waste haulage, materials handling, access and ventilation, vault access tunnels and waste emplacement rooms, and the emplacement boreholes into which the used-fuel containers will be placed.

The dominant radionuclide release mechanism is for groundwater to corrode the container and penetrate to the waste, leach out the radionuclides and carry them back to the surface. Potential flow paths from the containers to the biosphere include the excavations (rooms, tunnels and shafts), or when these are satisfactorily backfilled and sealed, excavation-disturbed zones that surround the excavations. Excavation methods that minimize damage to the rock are therefore preferred.

Located near Lac du Bonnet in southeastern Manitoba, Canada, the Underground Research Laboratory (URL) is being constructed in a previously undisturbed portion of a large granitic pluton to carry out experiments in a realistic geological setting pertinent to the Canadian nuclear fuel waste disposal concept. Two such experiments, the buffer/container and multi-component experiments[1], require full-scale emplacement boreholes (1240 mm diameter by 5 m deep) to be excavated in the floors of rooms in the URL. Discussions with mining, shaft drilling and diamond drilling contractors indicated that the necessary production equipment to drill the emplace ment boreholes underground was currently not available and therefore had to be developed.

This paper discusses the development of a prototype water jet coring machine to be used to drill the large-diameter boreholes.

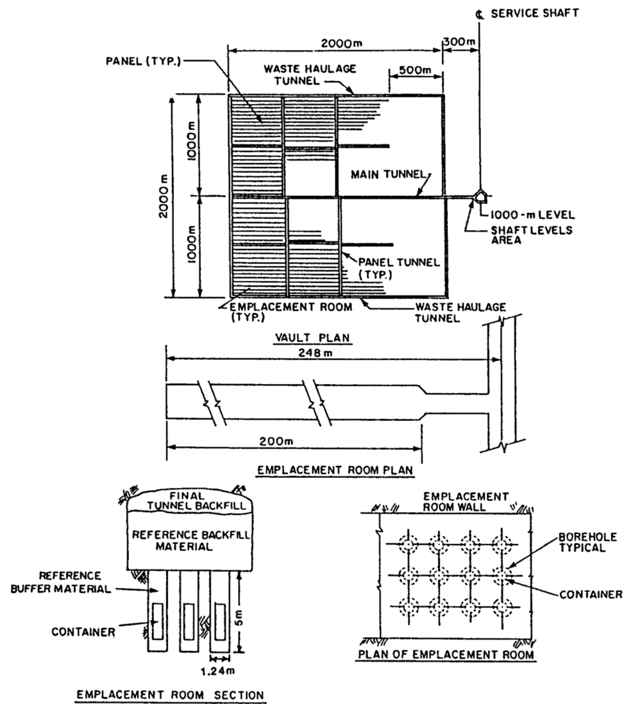

Fig. 1. Schematic diagram of nuclear fuel waste disposal vault.

DRILL DESIGN CONSIDERATIONS AND SPECIFICATIONS

URL rock type

The predominant URL rock type is pink to grey, massive, medium- to coarse-grained porphyritic granite[2]. Quartz diorite and amphibolite xenoliths, leucocratic pegmatite dikes, and quartz veins and inclusions occur locally in the main rock mass. A summary of mineralogical and mechanical propertiesof Lac du Bonnet granite is presented in Table 1.

Emplacement borehole and URL underground operational specifications

To meet requirements of the experiments, the tolerances for the borehole diameter and surface roughness were defined as 1240 mm +10 mm/−0.0 mm and ±5 mm, respectively. The axis of the bore hole was specified to be within 1° of vertical.

Design of the equipment must allow for transportation down the rectangular URL shaft (1.6 m by 2.1 m clearance) and underground through access tunnels (3.0 m high by 3.5 m wide). The mass of the equipment is limited to the 4.5 tonne capacity of the shaft hoist. The clearance in the test rooms limits the height of the drilling machine to less than 3.2 m, and so, in order to drill a 5 m deep hole, the drilling tool would have to be sectionalized. Moreover, if a coring technique is used, the core would have to be removed from the hole in pieces less than about 1.5m long to meet both spatial and shaft hoist capacity specifications.

Selection of drilling technique

After a review of potential shaft drilling, drilling/reaming and coring techniques, a coring technique that is an extension of high pressure rotary water jet rock slotting[3,4] was selected for development. Apparent advantages of the water jet technique over the other methods include: the water jet cutting nozzles are longer lived and much less expensive to replace than conventional cutting tools and bits, the equipment is relatively light and readily transportable, and, high thrust forces are not required for the water jet cutting nozzles. Moreover, the transient high pressure water forces should cause less disturbance to the surrounding rock than blasting, and perhaps even less disturbance than the vibrations and abrasive forces of conventional drilling machines. Also important was the potential longer term benefit of helping to develop a new rock cutting technology which might find application in other URL experiments and in a future disposal vault.

Background

Applications of high pressure water jets to mining, tunnelling and quarrying have been limited to the development of water jet-assisted tunnelling machines such as roadheaders and full-face boring machines[5,6], efficient water jet roof bolt drills[7] and water jet rock slotters for use in quarrying[4]. To the authors' knowledge, the extension of water jet rock slotting technology to drill large-diameter boreholes and shafts in hard crystalline rock is a unique application of this technology.

There are three types of water jets used for cutting rock: continuous water jets, cavitating or interrupted water jets, and water-abrasive jets, (or, simply

Table 1 Mineralogy and Mechanical Properties of Lac du Bonnet Granite[2]

Average		
Mineralogy	quartz—31%±4% plagioclase—38% ± 6% microline—27% ±7% muscovite—0.5%± 0.3% biotite—3.5% ±1.5% others—0.5% ±0.5%	
Bulk Density (Mg/m^3) 2.63±0.05		
Compressive Strength* (MPa)		
	Pink (to~260 m depth)	
	range	134–248
	mean	200
	Grey (below~260 m depth)	
	range	147–198
	mean	167

* Values representative of Underground Research Laboratory site

abrasive jets) in which hard particles such as garnet or quartz sand are added to a water jet to allow harder materials to be cut and deeper penetration of the jet. The selection of a particular type of jet for rock slotting depends on the rock type, as well as the intended application. Particularly important to water jet penetration and cutting is the porosity, grain size and bonding forces between mineral grains. Generally, medium- and coarse-grained rocks like granite and sandstone are readily cut with water jets without abrasives.

In drilling large-diameter boreholes and shafts with rock slotting technology, it is envisaged that two types of slots would have to be cut: a vertical, circular slot, the outside diameter corresponding to the specified borehole diameter, and a horizontal slot or notch in the core to facilitate core breakage.

A rock slotting study carried out in a granite quarry by Hawrylewicz et al.[4] indicated that a rotary, double water jet nozzle (i.e. a nozzle with two axially symmetric water jet orifices mounted on the end of a rotating high pressure water lance) could cut the required vertical, circular slot. The prototype rock slotter used for these tests was able to cut a straight slot, 5 m long, 3. 4 m deep and about 44 mm wide, with an effective exposure rate of 1.15 m^2/h (i.e. the rate of newly opened planar surface area). However, these tests also showed that the cutting effi ciency in a closed slot was reduced when water collected in the slot and the nozzle became submerged, and also when sand-sized cuttings collected at the base of the slot. Thus, the removal of both water and cuttings from a closed slot is an important design consideration.

Given the above, the following concepts for drilling large-diameter boreholes were established:

1) Design the coring rig to cut a circular slot using rotary water jets. The slot width should be at least 40 mm, the outside diameter corresponding to the specified borehole diameter. A rotary, double water jet was considered to be the most efficient for slotting the medium—to coarse-grained Lac du Bonnet granite. To allow cutting through pegmatite dikes, quartz veins and inclusions, and other features that might not be readily cut with the water jet, a rotary, abrasive jet should be developed as a backup to the water jet.
2) Design a horizontal notching jet to facilitate core breakage. An abrasive jet should allow a deep notch to be cut without inserting the nozzle. The core can be broken by either mechanical wedging in the kerf or by emplacing expanding cement grout in a sub-horizontal notch. Since the core pieces being removed from the hole should be no longer than about 1.5 m,

giving a length to diameter ratio of about 1.3, the expanding cement option for core breakage was considered preferable to mechanical wedging.

3) Design a simple core removal system. Conventional hoisting equipment should be adequate.

Nozzle design and development

Slotting tests were carried out on representative samples of Lac du Bonnet granite using the prototype rock slotter of Hawrylewicz et al.[4] The important parameters for rock slotting in granite with rotary water jets are water pressure and flow rate, nozzle rotation speed, nozzle traverse velocity and the downward incrementing of the nozzle per pass (step down). For the straight slotting tests described above, water was supplied to the nozzle at a pressure of 138 MPa and a flow rate of 76 L/min. The slotting lance was traversed along the slotting plane with an average velocity of about 100 mm/s, while rotating at about 300 rpm, and was stepped down in about 5 mm increments after each pass.

The objectives of the tests were to:

1) Assist nozzle selection and allow optimization of the nozzle designs for slotting and undercutting.
2) Develop the range of cutting parameters to yield the circular slot width and depth of undercut required to drill a borehole.
3) Allow estimation of coring and undercutting productivity.

Four nozzle types were tested:

– nonrotary, single abrasive jet undercutting nozzle.
– nonrotary, single water jet undercutting nozzle.
– rotary, double abrasive jet slotting nozzle.
– rotary, double water jet slotting nozzle.

The results of this testing program (Indescor Hydrodynamics Inc., unpublished data) are summarized in this paper.

The nonrotary undercutting nozzles were traversed along a rock sample at a constant stand-off distance of about 5 mm. The effect of the number of passes on undercut slot depth was assessed. The undercut slots were cut at a dip of about 5° to 10°. The rotary slotting nozzles were traversed along the rock samples and incremented down at 5 to 7 mm per pass. Multiple passes were performed in a vertical plane.

The undercutting nozzle tests showed that an abrasive undercut nozzle can cut a slot 100 to 120 mm deep without inserting the nozzle into the slot. The resulting slot is V-shaped in cross section, widest at the rock face and tapering into the rock. This depth of slot was achieved with 12 to 16 passes at a nozzle traverse speed of 700 mm/min. The water undercutting nozzle was not able to cut a slot deeper than 90 mm.

The rotary, slotting nozzle tests showed that both the rotary abrasive and rotary water double jet nozzles can cut slots to the required 40 to 50 mm width. The rotary water jets, however, were able to cut slots four to five times faster than the abrasive jets. This was attributed to deficiencies in the abrasives delivery system to the nozzles. An improvement in the abrasive supply system should increase the rate of advance.

Based on these tests, it was estimated that a 1240 mm diameter borehole could be cored in Lac du Bonnet granite at a rate of about 250 mm/h using a rotary, double water jet nozzle. To allow for potential variations in the cutting characteristics of dikes and inclusions within the granite and for the differences in cutting rate between the rotary water and abrasive jets, the coring rig should be designed so that the nozzle rotation rate can be adjusted from 100 to 300 rpm, the circumferential nozzle traverse velocity from 700 to 3000 mm/min and the downward increment from 3 to 9 mm per pass. The nozzles should operate at a pressure of 135 MPa and a water flow rate of 50 L/min.

The optimized designs of the rotary, double water jet slotting nozzle, the rotary, double abrasive jet slotting nozzle and the nonrotary, single abrasive jet undercutting nozzle are shown schematically in Figure 2. The rotary, double water jet nozzle is rotated by a pipe that also supplies high pressure water to the nozzle body. Two jets are created by two water orifices in the nozzle body. The rotary, double abrasive jet nozzle has twin orifices similar to the rotary water jet nozzle. The water jets suck abrasives through the abrasive feed line to create an abrasive slurry that is focused on the rock surface by the twin orifices. The nozzle is rotated by a high pressure water pipe while the abrasives are supplied through a hose to the nonrotating nozzle manifold.

Design and operation of the water jet coring system

A system to meet the anticipated rock and underground operating conditions has been designed and fabricated. The system comprises a coring rig, a high pressure pump unit, and an hydraulic power pack.

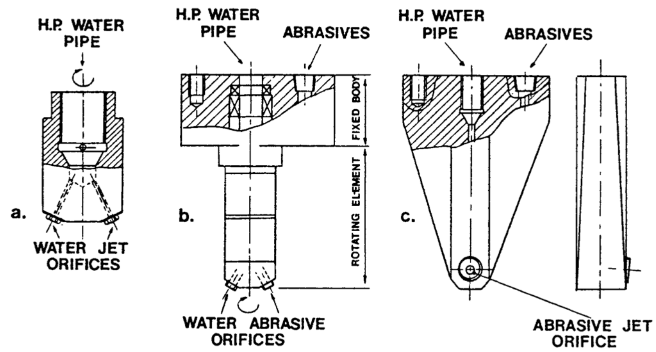

Fig. 2 Slotting and undercutting nozzles:

a. rotary, double water jet slotting nozzle.

b. rotary, double abrasive jet slotting nozzle.

c. nonrotary, single abrasive jet undercutting nozzle.

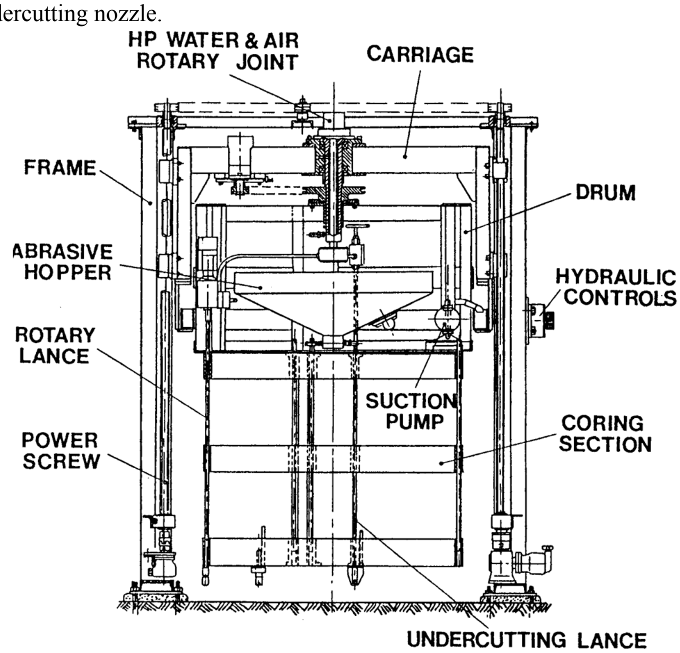

Fig. 3 Coring rig.

The coring rig, shown schematically in Figure 3, consists of a three piece frame, a carriage with a rotating drum and coring sections bolted to the base of the drum. The drum is rotated in relation to the carriage by a hydraulic motor and chain drive. The carriage can be moved vertically by power screws driven by a directly coupled hydraulic motor. The step down motion for coring is controlled by a second hydraulic motor driving the power screws through a gear drive. Both power screws are connected by a chain drive at the top of the frame. The carriage is guided by four lineal bearings mounted on the side frames. The vertical carriage motions and the drum rotation can be adjusted with hydraulic controls installed on the coring rig.

Due to the limited underground headroom, each coring section was made 1.33 m long. Thus, to core a 5 m deep hole, four such coring sections need to be connected together successively as the hole is advanced. Two rotary slotting lances, two undercutting lances and a suction cuttings removal pipe are attached to each coring section. Each rotary lance consists of a high pressure rotary coupling rotated by a pneumatic motor, high pressure pipe sections and couplings for adding additional coring sections and a rotary, double water or double abrasive jet nozzle. Two undercutting lances are placed diametrically opposite each other on the coring section to increase the cutting efficiency and to counterbalance the small horizontal reaction forces (~ 240 N) generated by the high pressure jets. These nonrotating, undercutting lances consist of high pressure pipes and a single abrasive jet nozzle. The rotary and undercutting lances are connected to the high pressure rotary joint (see Figure 3) with high pressure flexible hoses. The supply of high pressure water to either the rotary water or rotary abrasive nozzle for slotting, or the nonrotary abrasive nozzles for undercutting, is controlled by manual on-off valves. The 20 mm diameter cuttings removal pipe is connected to a pneumatic diaphragm suction pump mounted on the drum.

The rate of rotation of the rotary lances and the suction pump speed can be regulated by pneumatic controls installed inside the drum. The abrasive supply to the nozzles is induced by a vacuum created inside the abrasive nozzles. In order to assist smooth flow of the abrasive, a pneumatic vibrator is installed on the abrasive hopper.

Water is supplied to the nozzles by a high pressure pump unit. The pump unit consists of a Hammelman triplex plunger pump powered by a nominally 250-hp electric motor, low pressure water filters, hydraulic water accumulator and electrical controls. This pump is capable of delivering up to 56 L/min of water at a maximum pressure of 140 MPa.

Proposed coring and core removal procedures

After being levelled in the drilling location, the coring rig is anchored to the rock surface or a concrete floor or pad. The coring is initiated with the rotary, double water jet nozzle (Figure 4a) and continues to a depth of 1.3 m, the limit of cutting with the first coring section. At this point, an undercut slot is cut with the undercutting abrasive nozzles (Figure 4b). The proposed method to break the core is to emplace an expanding cement grout into the undercut slot and allow it to cure. Scaled tests in the laboratory using 200 mm diameter samples of Lac du Bonnet granite indicated that the core can be broken within 12 to 18 hours. To remove the broken core from the hole, the coring rig is moved off the hole and a suitable hoist is used to lift out the core.

There are two options to drill the required 5 m deep borehole:

1) Core the entire 5 m deep slot and undercut the core every 1.25 m without moving the coring rig off the hole. After the slotting is completed, the coring rig can be moved off the hole and the core can be broken and removed.
2) Core to a depth of 1.3 m, cut an undercut slot, break and remove the core before proceeding with further coring. The hole would be advanced following this sequence to a depth of 5 m.

The first option requires less movements of the coring rig and core removal gantry, whereas the second option would give better access to the bottom of the slot as coring proceeds. The field trials would indicate which option would be the most suitable.

TESTING THE PROTOTYPE WATER JET CORING EQUIPMENT

Three stages of testing the equipment were planned:

1) Shop tests on large granite blocks of Lac du Bonnet granite to assess the operation of the coring system, the quality and dimensions of a short cored hole and the effectiveness and efficiency of the undercutting nozzles in cutting undercut slots.
2) Surface trials at the URL to develop the technology to drill a 1240 mm diameter borehole 5 m deep. Equipment modifications and access to ancilliary equipment and machine shops were simpler on the surface for this stage of development than underground.
3) Underground trials at the URL to act as the final stage of development for the prototype machine.

The first two stages of testing have now been completed.

Shop tests

Shop tests were carried out on two block samples of medium- to coarse-grained, pink porphyritic Lac du Bonnet granite containing pegmatite veins. Both samples were 2.08 m long, 1.40 m wide and 0.55 m thick. These dimensions allowed the coring rig to be anchored to the sample as shown in Figure 5.

The first sample was used to test the operation of the coring rig and to optimize the nozzle rotation rate, traverse velocity and downward increment per pass for both the rotary water and rotary water-abrasive jet nozzles. The optimized system was tested using the second sample. The nozzles were operated with a water pressure of 135 MPa at a flow rate of about 50 L/min. Both samples were cored through and an undercut slot was cut in each core.

Results from the first sample confirmed the nozzle development tests and showed that the rate of coring with the rotary abrasive nozzle was significantly slower than with the rotary water nozzle. Using the abrasive nozzle, it was difficult to achieve the required slot width even with a lower cutting rate (see Table 2). The rotary abrasive nozzle required improvement, particularly an improved abrasive feed system.

The test on the second sample involved coring the first 0.445 m of the block with the rotary water jet nozzle and the last 0. 105 m with an improved rotary abrasive nozzle. The test parameters and the results of this test are also compiled in Table 2. After

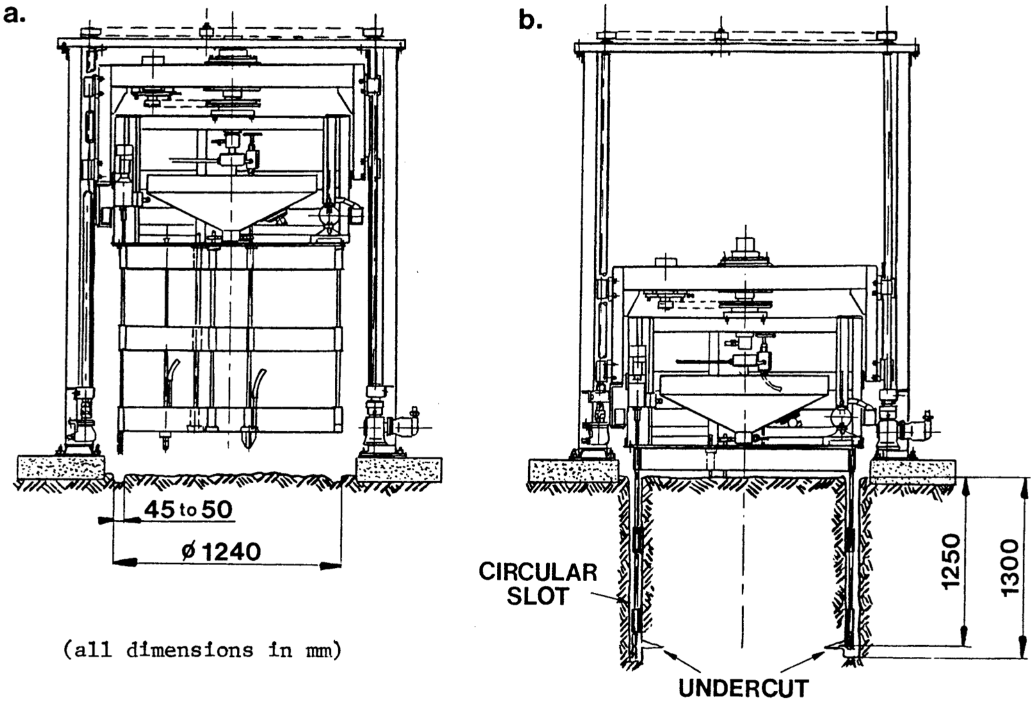

Fig. 4 Coring sequence:

a. initiate coring with rotary, double water jet nozzle.

b. undercutting core with single abrasive jet nozzles

coring and undercutting, the core was removed from the hole. Four diameters radially spaced at 45° were measured along the length of the borehole (see Table 3). As Table 3 indicates, the diameters are generally within the specified borehole tolerance for the section cut with the rotary water jet nozzle. Larger borehole diameters in the section cut with the rotary abrasive jet nozzle indicates that further testing and optimization of the cutting parameters is required for this nozzle.

The quality of the surface finish on the borehole wall is illustrated in Figure 6. The average of 26 readings of surface roughness on the borehole wall, including the pegmatite vein, was 2.5 mm with a maximum surface roughness of about 5 mm. The undercut slot cut in the core is shown in Figure 7.

The water and cuttings removal system was adequate for the shop tests. At depths less than about 0.40 m most of the cuttings were flushed out of the slot by the water jets themselves. The cuttings were

Table 2 Effects of nozzle rotation rate, nozzle transverse velocity and abrasives on the cut slot geometry and rate of cutting through Lac du Bonnet granite

Sample #/ test #	nozzle rotation	transverse velocity	av. depth per pass	av. width of slot	newly open surface	type of abrasive
	rpm	mm/min	mm	mm	m^2/h	
1/1	177	2500	6.5	52	0.975	none
1/2	177	2850	5.7	47	0.98	none
1/3	177	2850	7.7	43	1.31	none
1/4	254	1750	3.8	35	0.399	garnet
1/5	163	1750	3.0	37	0.315	garnet
1/6	undercut.	1100	9.5*	12	0.667	garnet
2/1	210	2800	6.35	45	1.07	none
2/2	undercut.	1030	4.6**	9	0.309	silica sand

* average for 10.5 passes
** average for 18 passes

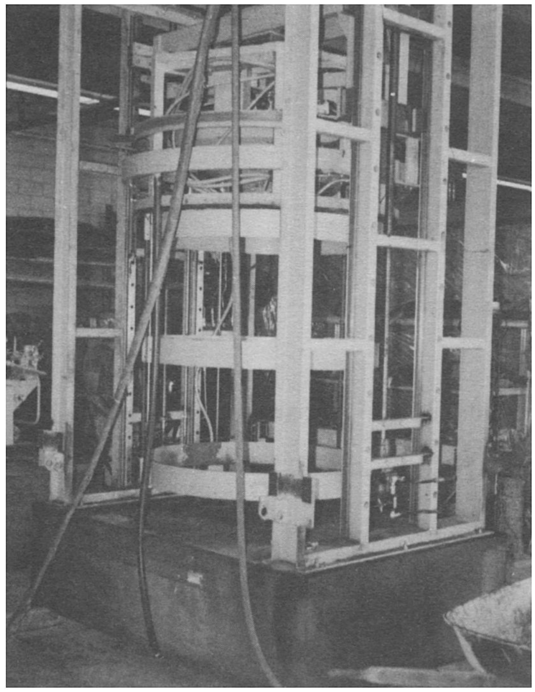

Fig. 5 Coring rig mounted on a granite block for shop test.

Table 3 Diameter Measurements; Shop Test Borehole #2

Depth Below Top of Block (mm)		Diameters at 45° Radial Spacing (mm)			Average (mm)
100	1245,	1243,	1242,	1244	1243.5
200	1246,	1245,	1244,	1246	1245
300	1248,	1246,	1245,	1246	1246
400	1250,	1250,	1248,	1250	1249.5
425	1254,	1251.5,	1249.5,	1250	1251
475	1258,	1251.5,	1253,	1252	1253.5
500	1256,	1252,	1254,	1255	1254

Note: Rotary water jet nozzle used to a depth of 445 mm; rotary abrasive jet nozzle used from a depth of 445 mm to the base of the sample.

predominantly medium-sized sand. Momentary jamming of the coring section occurred occasionally. This was attributed to coarser particles jamming between the coring section and the walls of the slot.

Major results of the shop tests were as follows.

1) A borehole meeting the specifications may be cored with the water jet coring rig at a rate of about 250 mm/h with a rotary water jet nozzle.
2) The rotary water jet nozzle can successfully core through granitic pegmatite veins while maintaining the required borehole diameter and surface roughness specifications.
3) Undercut slots 80 to 100 mm deep are achievable after 12 to 18 passes of the nonrotating, undercutting, single abrasive jet nozzles. Garnet sand is a more effective abrasive than silica sand.

Surface trials at the URL

The surface trials were conducted on an outcrop at the URL site. Two 6 m deep, HQ-sized pilot holes were drilled 3 m apart on the outcrop to define the lithology and potential fracture patterns in the rock before water jet drilling started. The core samples indicated the rock was generally a pink, coarse grained porphyritic granite. Several discontinous granitic pegmatite

Fig. 6 Shop test #2 sample.

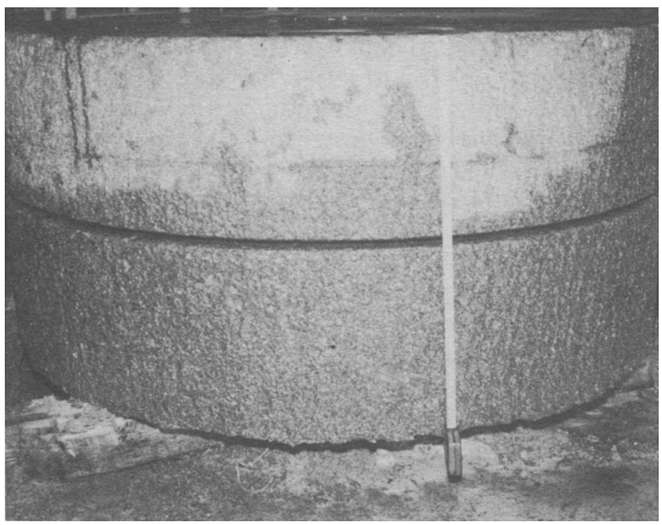

Fig. 7 Undercut slot cut in core of shop test #2 sample

zones 0.2 to 0.4 m thick were observed in the core samples. Sporadic quartz veins generally less than 15 mm thick were associated with these pegmatites.

The coring rig was set up and anchored on two concrete pads (Figure 8) axially over one of the pilot holes with the intent to drill a 5 m deep borehole concentric with the pilot hole. The preferred drilling sequence at the outset was to core to a depth of 5 m, cut the undercut slots, then break and remove the core.

The rotary water jet nozzle was to be the main cutting nozzle, with the rotary abrasive nozzle to be used only if considered necessary. The "target" cutting parameters were: nozzle rotation rate of 200 rpm; traverse velocity of 2600 mm/min, giving one revolution of the coring section in 1.5 min; and downward increment rate of 4 mm/min, giving an anticipated coring productivity rate of 240 mm/h. Coring went smoothly to a depth of about 0.47 m, where the cuttings removal system, including both the suction pipe and suction pump, began plugging. This depth closely corresponded to the top of a pegmatitic zone with quartz veins noted in the core samples. When the cuttings removal system was cleaned, gravel-sized pieces of quartz were found. Besides plugging the cuttings removal system, the remnants of the quartz veins were protruding into the slot and causing the coring section to jam periodically; this was overcome by recutting the irregular sections of the slot.

Below 0.47 m, the cuttings removal system became increasingly inefficient at removing gravel-sized cuttings breaking from both quartz veins and natural fractures in the rock. A manually operated "vacuum cleaner system" comprising a 25 mm diameter suction tube, a large sand and gravel trap and a 50 mm suction diaphragm pump proved successful in cleaning the base of the slot. However, although the coring production rate was approximately 250 mm/h, the rate of advance of the hole became so slow that cutting was discontinued at a depth of 1.74 m because of the need for frequent stoppages to remove cuttings from the base of the slot. The borehole diameter ranged from about 1250 to 1255 mm between the surface and a depth of 1.1 m.

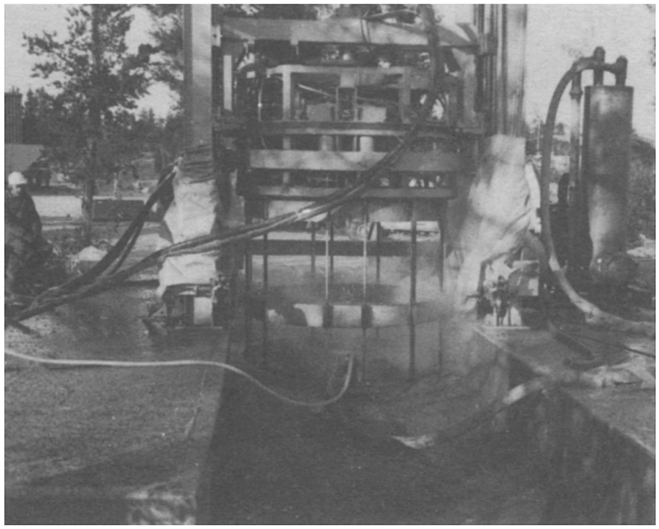

Fig. 8 Surface trials; note pilot hole suction system.

To overcome the difficulty in removing cuttings, the slot for a second surface trial emplacement borehole was drilled to overlap the second 96 mm diameter pilot hole that was used as a sump to collect the cuttings. In this case, cuttings were removed from the sump via a pipe connected through a sand and gravel trap to a 50 mm suction diaphragm pump (see Figure 8).

The revised cuttings removal system generally worked well. The hole was cored to a depth of 2.6 m at a coring production rate of about 190 to 240 mm/h using the rotary water jet nozzle. At times, because protruding remnants of quartz veins caused the coring sections to jam, advancement of the borehole was slow; sections of the slot had to be reamed with the water jet. Slowing the downward increment rate from 4 mm/min to 3.2 mm/min improved the performance somewhat by allowing a slightly wider slot to be cut. Jamming difficulties, however, still required selective reaming of the slot. At a depth of 2.6 m the sump hole deviated from the cored slot and coring was terminated due to an inability to remove the cuttings.

Natural, subhorizontal fractures at a depth of 1.4 m in the first borehole and 2.0 m in the second assisted the core removal process. The 1.4 m length of core in the first hole was broken vertically into four pieces by drilling an array of 25 mm diameter holes and filling them with expanding cement. The core broke within 18 hours. As an additional test of the effectiveness of the expanding cement, the core from the first shop test sample was broken by applying the expanding cement grout in the undercut slot.

Summary of tests

The shop tests and surface trials indicate that the prototype water jet coring rig can likely be used to drill a borehole 1240 mm in diameter 5 m deep in Lac du Bonnet granite. Moreover, although protrusions of remnants of quartz veins at times caused some of the coring sections to jam, the rotary water jet nozzle was able to cut through them. The coring production rates of about 190 to 250 mm/h generally conform to those predicted from the nozzle development tests. The slower rate gave some improvement in performance where jamming of the coring sections occurred. Moreover, tests with expanding cement during the field trials have shown that the core can be either broken horizontally, using the undercut slot, or vertically. The broken segments of core can be readily removed from the borehole.

Experience from the surface trials indicates that it would be preferable to remove the core in stages as the hole is advanced rather than to attempt to slot the entire 5 m before removing the core. This option not only allows access to the base of the slot as the hole is proceeding, even at depth, but also minimizes potential problems with cuttings removal and jamming of the coring sections. Underground trials at the URL using this procedure are now in progress and will be completed by May 1989. Experience gained from these trials will allow further assessment of the operation of the coring rig and could lead to modifications to improve system performance.

CONCLUSIONS

The stages of development of a prototype water jet coring rig to drill large-diameter boreholes in granite have involved nozzle testing and optimization, shop trials in large granite blocks and field trials at the URL. Each stage has provided information relevant to the operation of the coring rig and the successful com pletion of a 5 m deep emplacement borehole. The major difficulties to date have been associated with cuttings removal and the jamming of the coring sections during slotting. Underground trials at the URL, which are now in progress, will allow further assessment of the operation of the water jet drilling system. The following conclusions can be made regarding the drilling of 5 m deep, 1240 mm diameter boreholes in granite.

1) A rate of coring production of about 190 to 240 mm/h can be achieved using rotary, double water jets operating at a water pressure of 135 MPa and a water flow rate of 50 L/min.
2) An 80 to 100 mm deep undercut slot can be cut in the core in about 35 min using two nonrotary abrasive single jets operating at a water pressure of 135 MPa and a water flow rate of 50 L/min. About 3 kg/min of garnet abrasive feed is required.
3) An 80 mm deep undercut slot is sufficient to break the core with expanding cement.
4) The preferred method to advance the borehole to a depth of 5 m is to remove the core as coring of the borehole proceeds. This will allow more direct access to the slot as drilling is proceeding.

ACKNOWLEDGEMENTS

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References

1. Kjartanson, B.H. and Gray, M.N.Buffer/container experiments in the underground research laboratory. In: Proceedings of the 40th Canadian Geotechnical Conference. Regina, Saskatchewan, Canada. 1987. p 275–283.
2. Katsube, T.J. and Hume, J.P.Geotechnical studies at Whiteshell Research Area (RA-3). CANMET. Mining Research Divisional Report MRL 87–52. 1987.
3. Vijay, M.M., Remisz, J., Hawrylewicz, B. and Puchala, R.J. Considerations in the use of high speed water jets for deep slotting of granite. In: Proceedings of the 4th U.S. Water Jet Conference. University of California, Berkeley, U.S.A. 1987. p 121–128.
4. Hawrylewicz, B., Vijay, M.M., Remisz, J. and Paquette, N. Design and testing of a rock slotter for mining and quarrying applications. In: Proceedings of the 9th International Symposium on Jet Cutting Technology. Sendai, Japan. 1988.
5. Morris, A.H. and Harrison, W. Significant advance in cutting ability—roadheaders. In: RETC Proceedings. vol. 1, chapt. 20. 1985.
6. Baumann, L. and Heneke, J. High pressure water jets aid TBM's. In: Tunnels and Tunnelling. vol. 13, no. 1, Jan.-Feb. 1981. p. 21–26.
7. Reichman, J. Optimization of water jet systems for mining applications. In: RETC Proceedings. vol. 1, chapt. 19. 1985.

Shaft sinking by ground freezing in the coal-mining industry in the Federal Republic of Germany

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SYNOPSIS

A survey on the development of shaft sinking techniques in non-stable water-bearing strata in West German coal mining is given. For more than 100 years ground freezing has been successfully used to provide temporary stabilization. The bedded tubbing lining widely used, is in Germany being replaced by a so-called sliding lining. The sliding lining is characterized by a bedded outer lining (i.e. concrete block system) and an inner lining (i.e. composite structure) supported on a foundation, seperated by an annulus filled with an asphalt mass. The advantages to mining operations within the shaft pillar and the creation of an absolutely watertight shaft construction are demonstrated. This lining system requires during planning the ground freezing specific boundary conditions as well as for the sinking stage. New R&D results during the last three decades concerning the freezing method will be illustrated by examples of different freeze shaft projects with sliding lining. The different lining systems for watertight sliding shafts will be compared to each other with regard to construction, engineering and structural analysis. Relative to the thickness of the over-burden stratum a simple concrete lining together with a thin waterproof steel liner is sufficient, with increasing freezing depth composite structures of concrete and steel and/or cast iron are necessary. In addition to the longitudinal and the transverse stability of the shaft, the structural design of the inner composite lining is very important. Some special problems of lining design for deep shafts from a civil engineering point of view will finally be discussed.

INTRODUCTION

In the West German coal industry, surface-connecting shafts may be up to 1600 m in depth. When sinking through a stable overburden, concrete is used as a strata-bedded shaft lining. In areas with a water-bearing, unstable overburden, the initial construction work is carried out using the ground freezing method [1]. Excavations are first made within the protection of the freeze circle, using an external concrete block lining down to a foundation level in load-bearing ground. The inner shaft casing, which rests on an annular footing just like a chimney, is then constructed. It is preferable for the annular gap between outer lining and shaft casing to be filled with liquid asphalt so that any deformation of the strata due to mining does not damage the casing. When the lining has been installed up to the shaft collar, the freeze plant is shut down and the plug of frozen ground around the shaft allowed to thaw. A steel casing, with watertight welded joints, surrounds the inner lining to prevent the subsequent ingress of water. These so called sliding shafts thus act like stable pipes floating in a fluid (Figure 1).

The first sliding shafts, which were constructed some 30 years ago[2], already featured composite linings, which comprised riveted or bolted U-section steel rings with a non-reinforced intermediate concrete fill. Between 1957 and 1960 the Wulfen nos. 1 and 2 shafts[3] were the first to be fitted with welded steel plates. Be cause of the uncertainty in assessing the forces acting on the lining as a result of coal winning close to the shaft, it was assumed that in exceptional cases failure of one of the outer plates would allow the full weight of the water pressure to act on the internal steel lining cylinder. To protect against radial stress it was nececcary to provide considerable anchorage of the inner casing, and at some points to establish a connection with the outer casing. In the North Shaft (1964–1967) pressure-relief pipes were incorporated in the inner steel cylinder[4], thus abandonning the concept of an inner lining designed to withstand the full water pressure. A load rating of 50% of the water pressure was retained for exceptional circumstances. Beginning with Auguste Victoria no.8 shaft[5], which was completed in 1966, sliding shafts are now usually constructed from steel and concrete without an inner casing. The watertight welded outer steel casing is normally 8 mm thick, while the steel concrete lining may be as much as 110 cm in thickness. The steel outer plates have never failed to provide complete watertight protection. Most sliding linings have a maximum construction height of 300 m from the foundation, and a simple concrete lining may be used in the non-aquiferous stable ground below the foundation. In the case of the Voerde sliding shaft [6], which passes through different strata beds to a depth of 600 m, a completely new type of shaft lining had to be designed. A structure composed only of reinforced concrete with a welded steel casing of only 8 mm plate thickness, as used in previous shafts of this kind, would in this case have resulted in uneconomical

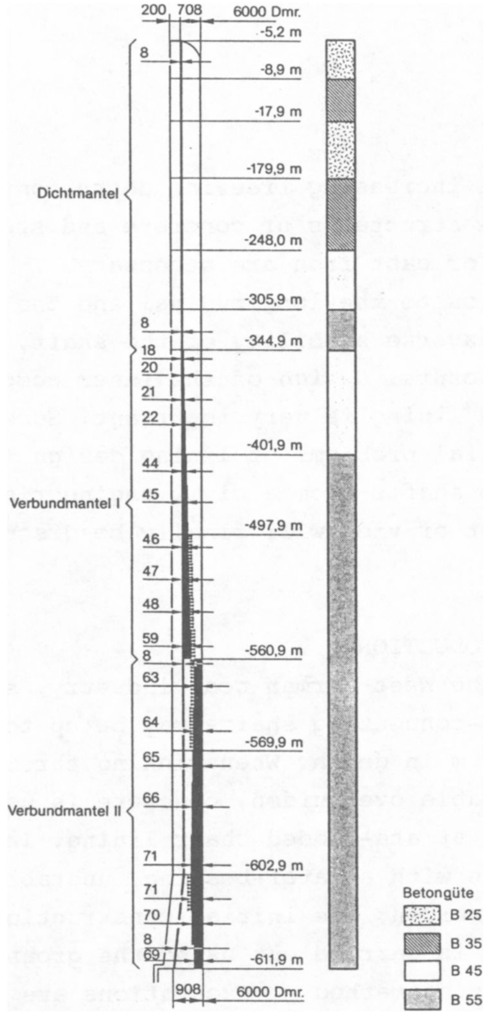

Figure 1. Inner lining of Voerde freeze shaft[6] (composite lining I to depth of 560.9 m composite lining II to shaft foundations)

wall thicknesses for the steel concrete in view of the high petrostatic pressures from the asphalt mass. For this reason the outer composite casing was provided with reinforcement which increased with shaft depth, and in the lower section of the shaft, an inner composite casing was introduced. The inner composite casing, or liner, acts like armouring and by reason of its higher coefficient of elasticity reduces the required wall thickness of the steel concrete structure. When employing liners, the outer tight-welded steel casing is still needed to ensure water-tightness. While weldable steel grades are used exclusively for the outer casing, high-grade cast iron grades, made from spheroidal graphite cast iron (GGG), are available, as a substitute for the steel, for the construction of the liners. The second deep sliding shaft, some 560 m in depth, was sunk at Gewerkschaft Sophia Jacoba[7] using for the first time an orthogonal braced GGG 50 liner (Figure 2).

Given the geology of the Ruhr Carboniferous formations, the thickness of overburden increases as mine workings move further north-wards. In the water-bearing, unstable strata encountered here the shaft support components for future ventilation and winding shafts can, beyond a depth of about 400 m, no longer be economically manufactured from homogeneous mate rial. At great depth it is necessary to use mainly composite shaft support structures made from cast iron or steel and concrete. In this respect two basic designs are possible

1. The outer steel casing, which is a watertight, welded structure, is designed as an “outer composite casing” whose thickness increases with greater depth; an inner liner then becomes unnecessary. The enclosed concrete must be provided with appropriate reinforcement. Alternatively,

2. The outer, watertight, welded casing is designed as a thin, continuous steel skin, with an inner composite casing also being used. Since the inner liner does not need to be water-tight, various iron-based materials can be employed in its manufacture. Because of the favourable multi-axial pressure situation, the concrete between the iron casings does not usually require reinforcement.

Figure 3 shows the quantity of steel required for both types of shaft lining given specific concrete wall thicknesses and asphalt densities. With regard to steel consumption, it can be seen that the associated curves intersect at a depth of about 450

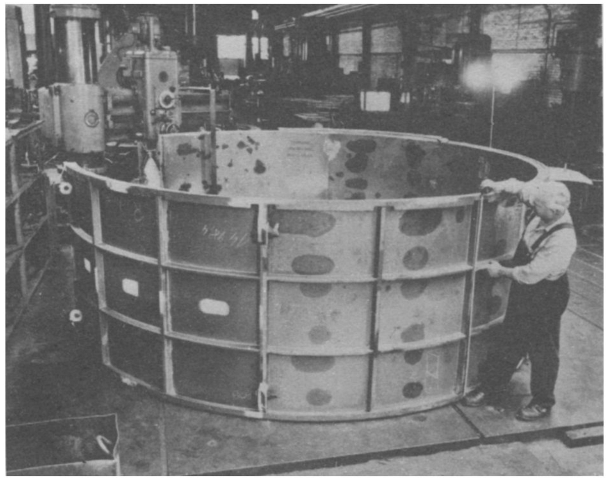

Figure 2. Pre-assembly of liner ring (Buderus AG, Wetzlar)

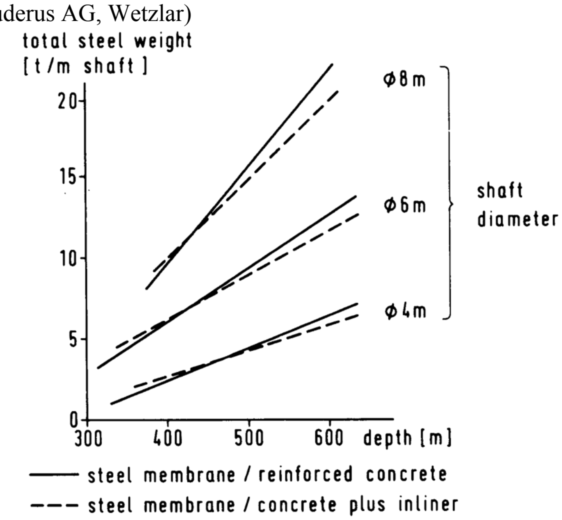

Figure 3. Quantity of steel used for composite shaft linings

m, irrespective of the inside diameter of the shaft. At shaft depths of between 400 and 500 m, the two support designs are practically equal in terms of the quantities of steel needed; at greater depths the system which uses an inner liner becomes much more favourable. Since calculation rules for shaft sinking state[8] that with increasing depth the uniform horizontal pressure becomes the decisive measurement quantity, as opposed to other load conditions, it is advantageous to employ a design variant in which the more rigid support elements are concentrated around the inner periphery of the shaft.

In the years to come, increasing importance will be attached to composite shaft support systems with liners when constructing deep sliding shafts. Indeed, for reasons associated with construction methods it may also prove logical to use a liner, which among other things serves as a casing, even at shallower depths. The theoretical aspects of stability[9] then assume greater significance, in addition to normal stress analysis.

STRESS AND STABILITY CALCULATION OF SLIDING SHAFT LININGS

Originating from ideas put forward by de Vooys[10], sliding shafts have now been constructed by the coal industry for 30 years to overcome the effects of mining near the shaft and to allow sinking through unstable, water-bearing overburden. The Rheinberg sliding shaft, which is currently under construction, is the 21st shaft of this type to be built (see table). In the sliding joint between the concentric cylinders of the impact-absorbing outer casing and the inner casing, the asphalt fill—composed of bitumen and limestone powder—creates a definite set of load conditions. The asphalt used for the majority of sliding shafts has a density of 1.3 g/cm^3 and serves to counteract the effects of external water pressure and horizontal strata load. This ensures uniform external compressive loading on the inner casing, while vertical relative displacement resulting from mining near the shaft is absorbed in a smooth and compatible manner. The viscosity of the asphalt fill is determined by selecting a suitable bitumen grade with the result that in the event of strata movement the shear stresses remain within tolerable limits.

Nr.	Schacht	Unternehmer	Bauzeit	ø(m)	Endteufe	Gefrierteufe
1	Auguste V.7	DB/WTS	56–59	6.75	960	230
2	Wulfen 1	HL	57–59	7.30	1076	270
3	Wulfen 2	HL	58–60	7.30	1076	270
4	Warndt	DB	58–60	7.50	750	348
5	Sophia J.6	DB	61–62	6.75	620	268
6	Auguste V.8	DB	63–66	6.75	1056	218
7	General 8.8	GK/GW/H	64–67	7.50	980	115
8	Nordschacht	HL/DB	64–67	7.30	1020	320
9	Altendorf	GK	67–69	5.00	837	115
10	Sophia J.7	DH/DTG/GHH	76–77	3.20	410	Bohrschacht
11	Prosper 10	GW	77–80	8.00	1070	140
12	Lauterbach	TS/GTG	78–80	7.00	950	220
13	An der Haard	DH/GK	78–80	8.00	1115	153
14	Polsum 2	TS	79–81	8.00	656	99
15	Haltern 1	DH/FK/GK	79–83	8.00	1135	217
16	Haltern 2	DH/FK/GK	80–83	8.00	1077	217
17	Voerde	GW	80–86	6.00	1060	581
18	Hünxe	TS	82–86	8.00	1370	327
19	Sophia J.8	K	84–87	4.00	930	558
20	Auguste V.9	DH	86–88	8.00	1330	210
21	Rheinberg	GW/TS/GTG	86-(91)	7.50	1300	526

Unternehmen (contractors):

DB	Deilmann Bergbau
WTS	Westrheinische Tiefbohr und Schachtbau
HL	Haniel & Lueg
GK	Gebhardt & Koenig
GW	Gewerkschaft Walter
H	Heitkamp
DH	Deilmann Haniel
DTG	Deutag
GHH	Gutehoffnungshütte
GTG	Gesteins- und Tiefbau
TS	Thyssen Schachtbau
K	Kopex
FK	Froelich und Klüpfel

Table
Die Gleitschächte im Steinkohlenbergbau
(The sliding shafts in coal mining)

When determining the inner-casing dimensions[11], the stress calculation for uniform and non-uniform horizontal loads is of decisive importance. The calculation sequence is laid down in the appropriate guidelines[8] for calculating shaft linings in unstable strata. According to these rules, the working stresses present are compared with the permissable stresses of the shaft construction materials. The stability calculation primarily takes account of ring buckling in the horizontal plane, wherein with increasing wall thickness, as characterised by the slenderness ratio of the structure (Figure 4), this effect becomes less important than the stress analysis. Slenderness ratios greater 50 are still only encountered with foreshafts of 100 m in depth. Those who drew up the guidelines could not have foreseen then that the characteristic length (length of sliding lining to outer diameter) would increase progressively in such a way, as a result of greater shaft depths. It is thus hardly surprising that uplift stability, which is an increasingly important load condition, is not mentioned until dealing with deep sliding shafts.

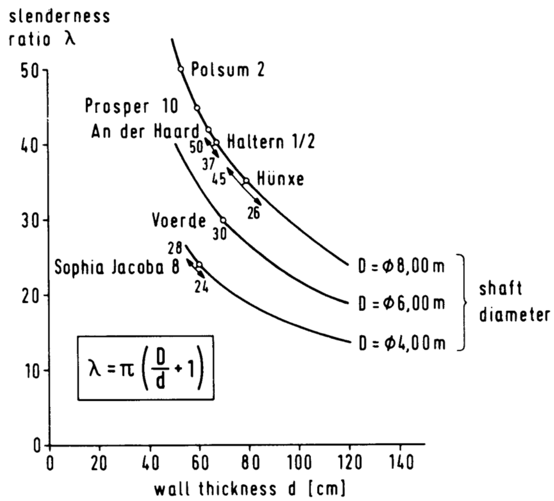

Figure 4. Slenderness ratio of various sliding shafts

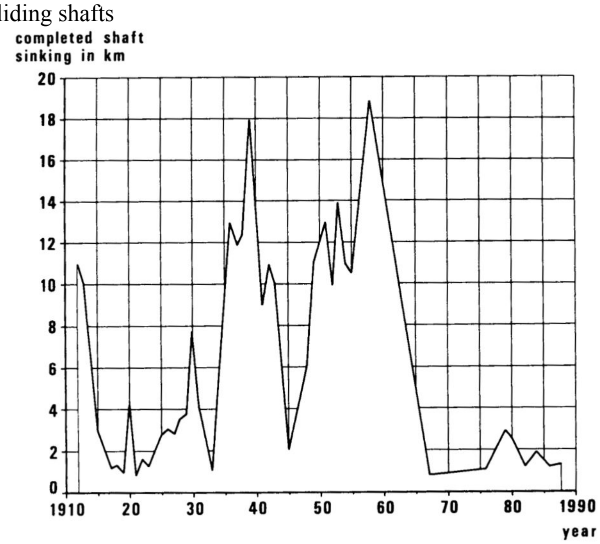

Figure 5. Shaft sinking in Germany

In retrospect it can be claimed that the calculation guidelines for sliding shafts have proved their worth, having been employed in shaft sinking projects at home and abroad. The fundamental correlations established by H. Link, the founder of modern shaft engineering statics, which are set forth in[12] and in previous works, are still valid even today.

ENGINEERING DATA OF 10 FREEZE SHAFTS IN GERMAN COAL MINING

The tubbing method lost much of its importance in Germany, when high priority was given to mining operations near the shaft, and the trend was towards asphalt jointed sliding shaft linings not bonded to the strata. 20 sliding shafts have been constructed during the last 30 years.

In general shaft sinking development in the Federal Republic of Germany for mining has shown a steady regression (Fig. 5) over the past 30 years.

The only transient upwards trend, after the oil crisis, went downward again quite soon, and an increased need for new major shafts is not within sight. Among the current shaft sinking projects, those which require freezing up to 600 m are particularly prominent. I quote here the shafts VOERDE, SOPHIA-JACOBA No. 8, and the RHEINBERG shaft just under construction.

It is obvious that the complex structure requiring the placement of a concrete block lining up from the foundation level, and subsequently the construction of the internal lining similar to a chimney structure is correspondingly expensive. Sliding shafts, including freezing of the strata, are approximately threetimes as expensive (Fig.6) as a conventional shaft.

However they have the following advantages:

- mining operations within the shaft pillar are possible

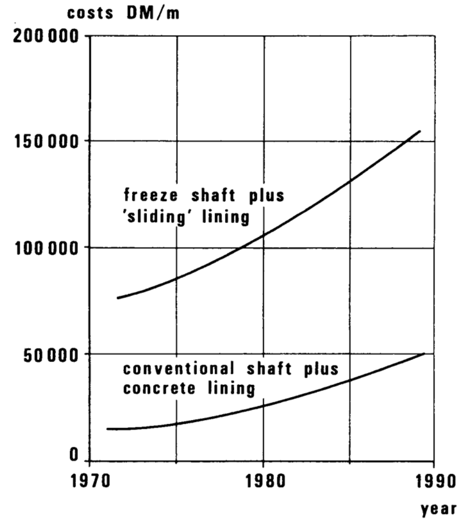

Figure 6. Costs for shaft sinking

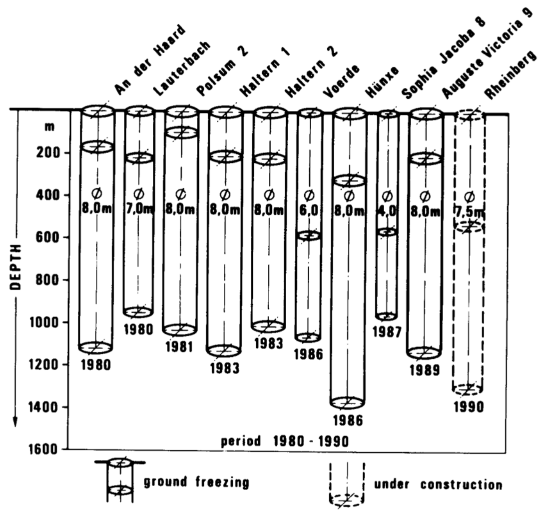

Figure 7. Freeze shafts in coal mining

– the external welded steel membrane guarantees absolute water-tightness.

Indeed in the case of several "sliding" shafts[13], working directly underneath resulted only in damage to the strata-bonded concrete lining of the shaft, and not in damage to the sliding portion of the lining.

Let us now take a closer look to the freeze shafts sunk in the West German coal mining industry (a total of 10 in this decade) (Fig.7).9 of these shafts have been already completed. For RHEINBERG shaft the hauling of the first kibble has started in December 1988. With a freezing circle diameter of 22 m this shaft is the largest sliding shafts ever sunk in Germany. Also in this case, the external lining installed during shaft sinking comprises shaped concrete blocks arranged over some length in several rows. This is the distinctive feature of other, even deeper freeze shafts sunk in Belgium, Canada and England where the freeze for the sliding systems needs to be maintained over a long period. For this reason comprehensive work has been carried out in the field of freeze shaft design, particularly over the past few years. The significance of this research work may be demonstrated less by formulae than by discussion of the geotechnical aspects and thermophysical boundaries.

Geotechnical aspects

To quote the example of a 500 m deep freeze shaft[14] the average strata conditions require about 10 m of frozen wall thickness. Using vertical freeze tube arrangement, this frozen body is cylindrical and necessarily overdimensioned in its upper part.

When looking at the frozen cylinder for elasto-plastic calculations this body is subdivided into two sections (Fig.8). The radial pressureis assumed to be induced by water and strata pressure onto the elastic thick-walled cylinder of the frozen body.

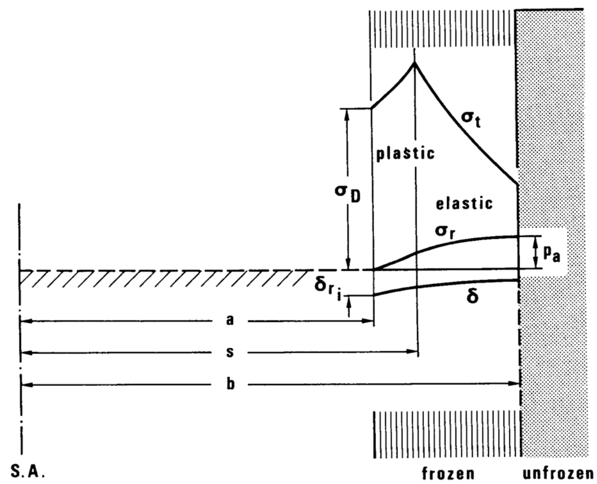

Figure 8. Stress and strain

The formation of an internal plastic zone is deduced by classical strata mechanics. As to the material properties, this zone must be described by an adequate flow rule. In this case, substantial differences are found among various dimensioning formulae known from literature[15]. On the internal rim of the frozen body, the uniaxial compressive strength of the frozen body is active. By introducing of a support resistance such as that provided by a wall of shaped concrete blocks—which creates an additional confining pressure—the stress pattern can be influenced.

When the permanent lining is installed immediately—using either tubbing or concrete—only the short-term strength of the frozen soil is of im portance. Cohesion essentially determines the strength of the material, and a friction effect can hardly develop. In case of an long-term maintenance using a shaped concrete block lining (for sliding shafts the scheduled period can be more than a year), however, the friction angle is of increasing importance, i.e. cohesion decreases with time while particle-to-particle friction within the structure of strata is activated.

For the determination of the soil-physical characteristics, triaxial compression tests are necessary. In this case the description of the shear strength according to TRESCA is not used but rather the known yield criterion as per MOHR-COULOMB. In many cases the long maintenance of the frozen body also requires time-dependent statements on the stress and strain history. For this purpose, creep tests are carried out. The variety of parameters to be catered for does not make design easier, whether by empirical methods or by rheological models.

The essential questions are centered on the assessment of radial convergence of the unlined shaft lengths[16] and the “aging” of the frozen material. The latter, of course, is again a function of the kind of external lining, and whether a soft lining made from shaped concrete blocks or a harder concrete panel lining is chosen. For daily shaft sinking work, in-situ measurements and comparative assessment against the calculated data are important. Most calculations assume constant external pressure and constant support resistance. However, rheological descriptions are also known.[17,18]

Thermophysical boundaries

In the following the freezing technological data of 10 freeze shafts in West German coal mining are compiled for statistics. The data clearly show the relation freezing circle D/excavation diameter A being a function of depth. This logically means increasing frozen cylinder thickness as a function of increasing depth. The average relation is D/A=1.2+T (m)/1000 (m). All projects carried out, i.e. the deepest shaft with 581 m of frozen length (VOERDE) as well as the shallowest shaft with 99 m of frozen length (POLSUM No. 2) are situated within this range of ±10 %. (Fig.9)

The comparison of the ground freezing installations is similar interesting. The performance of the refrigeration plants is seen to reach up to 15 GJ/h. The pipe capacity per 1 m of installed freezing pipe averages 600 kJ/h which corresponds to approx. 143 kcal/h. This value is still almost twice as high as the theoretically calculated requirements[19]; and means that a sufficient percentage of thermal losses is compensated by such capacity. On the other hand, the variation range of data was stated to be of ±20 % (when comparing the projects carried out). Obviously, the necessary rating depends essentially on the stratigraphic features, the soilphysical data, and in particular on the water content of the frozen strata (Fig.10).

When comparing the various freeze-pipe arrangements the picture looks more uniform again. Three of the German companies specialising in freeze shaft technology prefer mostly a spacing ranging between 1.2 m and 1.3 m so that for all of the 10 freeze shafts an arithmetic average of 1.28 m was calculated. A fourth specialist company prefers larger freeze-pipe spacing. Accordingly, the shafts VOERDE[20] and RHEINBERG go beyond this 10 % band-width. The total of the freeze-pipe installations recorded for three projects totals more than 20 km of length. With these considerable freezing depths the material

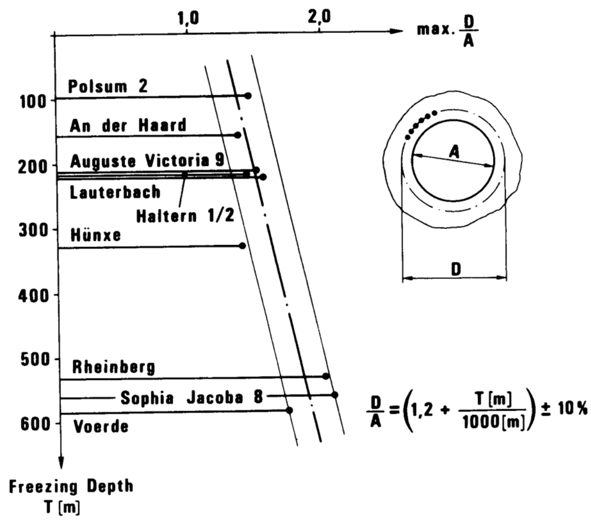

Figure 9. Freezing circle D/ excavation diameter A

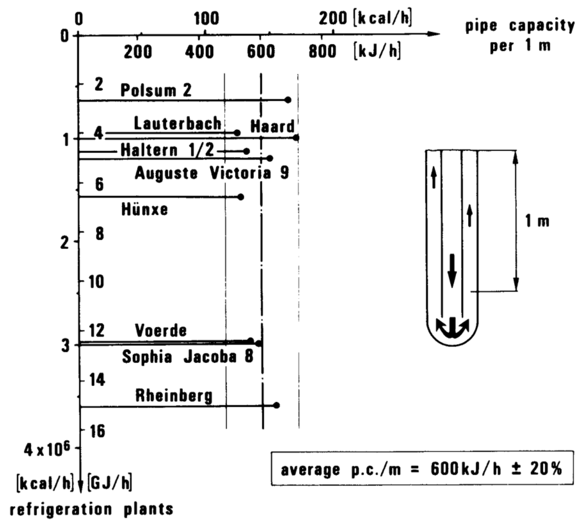

Figure 10. Ground freezing installations

choice for the casings and the connections of the pipes are of most importance. R & D carried out clearly has shown the unsuitability of API long-socket connections so that Ω-type connectors were preferred in the case of deeper freeze shafts (Fig.11).

EFFECTS OF MINING ON SHAFT LINING PERFORMANCE

When the sliding shaft lining is subjected to bending stress, the steel concrete rings can tilt against one another to create a lining cylinder with gaping joints. The high elasticity of the outer steel casing ensures that the inner lining remains water-tight up to and beyond its yield limit. Figure 12 shows a typical critical state of load caused by adjacent mining operations. Gaps have appeared at the joints of the concrete lining and the yield limit has been reached at points in the steel casing. This situation illustrates that such systems have a comparatively high capacity to absorb distortion created by mining operations. In such circumstances parts of the steel casing section are usually stressed beyond their limits of elasticity; this is justifiable when using materials which have a pronounced yield limit.

In the period 1979–1980 Auguste Victoria no. 8[13] shaft, whose sump depth at that time was 1037 m, was under-worked by a face in the 2.2 m thick D/C (EB: C/1/2/B) double seam at a depth of 1190 m. The shaft was loca ted in the seam horizon in the centre of an unworked troughed fault block some 400 m wide along the strike. The 300 m long coal face lay centrally to the shaft in its vertical projection. The face started up 500 m to the north of the shaft and moved to a line 800 m south of it (Figure 13).

The greatest stress suffered by the shaft was due to compression and stretching in the carboniferous and in the overburden below the 700 m level; the shaft itself was not put at risk (Figure 14). Small compression forces, which would then be

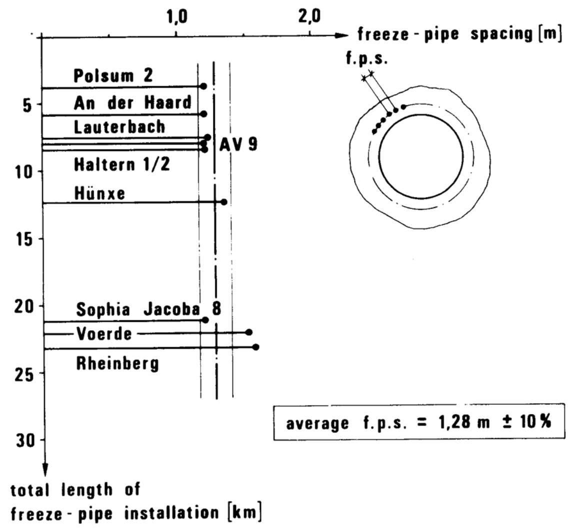

Figure 11. Freeze—pipe arrangements

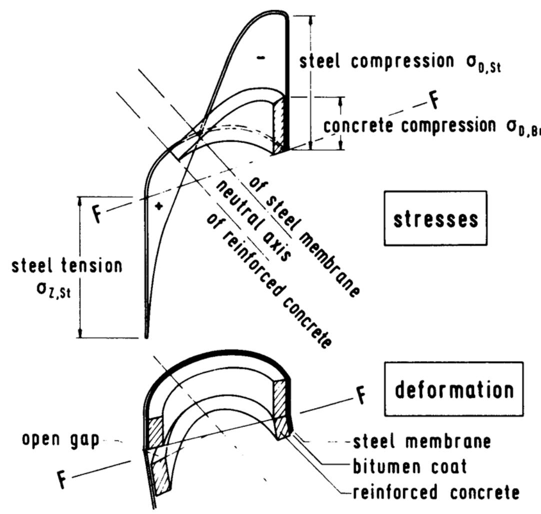

Figure 12. Bending stress acting on concrete lining with outer steel plate casing. Deformation (below) and stress in the steel and concrete structures.

dissipated by stretching action, were observed in the sliding section of the shaft, together with small amounts of tilt in the shaft axis. Compression had caused the asphalt level to rise by 3 to 4 m. Mining operations seemed to have little or no effect on the shaft foundations or on the immediate underlying strata. This zone is permanently subjected to vertical pressure from the weight of the inner lining and asphalt casing, even when subsidence of the strata occurs as a result of mining. The inner lining, shaft foundations and underlying bedrock apparently remained free from vertical pressure redistribution. The continuous effect of vertical loading means that strata break up is not possible in this area; however, the shaft lining can be affected by mining in the form of bending, stretching and compression forces.

CONCLUDING REMARKS

Auguste Victoria no. 7 shaft, which was sunk in 1956, was the first sliding shaft lining to be constructed by the German coal industry. Exhaustive studies by construction firms and con tinuous development work at Auguste Victoria[21] have produced today's successful shaft design. When mining is to be practised close to the shaft, there would appear to be no alternative design in sight to cope with the enormous stresses and to meet the requirement for absolute watertightness.

Assuming efficient use is made of the available materials, the inner lining can be calculated statically when fluid sliding compounds are employed. The guidelines issued by Steinkohlenbergbauverein[8] in this respect have proved reliable in the field. Increasing slenderness ratios and characteristic lengths have, however, brought new aspects to the fore. In addition to stability analysis in an annular and axial direction, greater attention should be paid to stress calculations with increasing lining wall thickness, particularly in the case of composite linings. The question of inherent stress, e.g. as a result of temperature

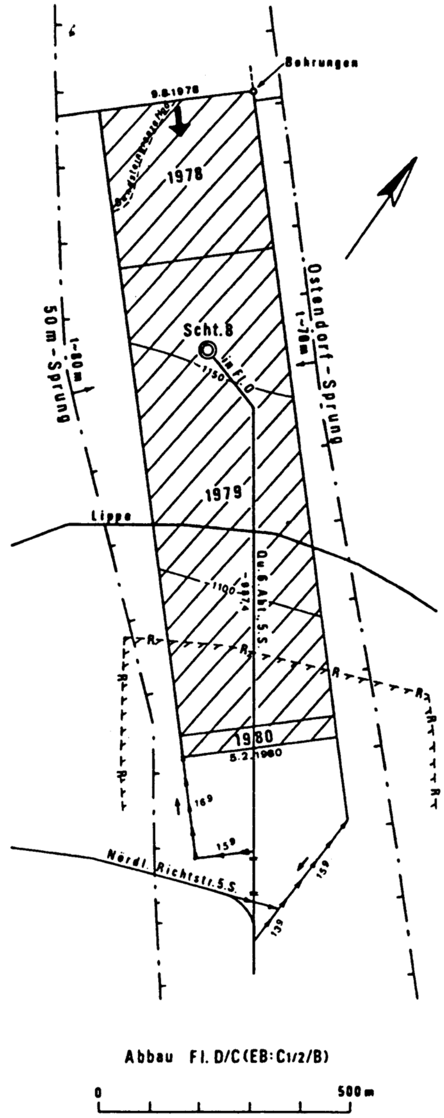

Figure 13. Plan of workings beneath Auguste Victoria no.8 shaft (longitudinal section)

gradients or structural imperfections, may also gain greater significance in shaft design as construction components continue to be perfected.

In the years to come, concrete will continue to be the dominant shaft lining material, with current grades of high quality concrete already being included to a large degree because of their high permissable compressive strength. Shaft linings, particularly in unstable, water-bearing strata, are subject to planning approval from the Mines Inspectorate[22]. In this respect, the planning of individual projects with collaboration between the relevant Mines Inspectorate, consultant experts, the clients and contractors, has always resulted in a productive consensus.

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References

1. KLEIN, J. (editor) (1985): “Handbuch des Gefrierschachtbaus im Bergbau.” Glückauf-Betriebsbücher 31, Glückauf, Essen
2. JANSEN, F; GLEBE, E. (1959): “Shaft Sinking in West German Coal Mining Industry.” Symposium on Shaft Sinking and Tunnelling, The Institution of ning Engineers, 15–17. July 1959, London, Paper No 6

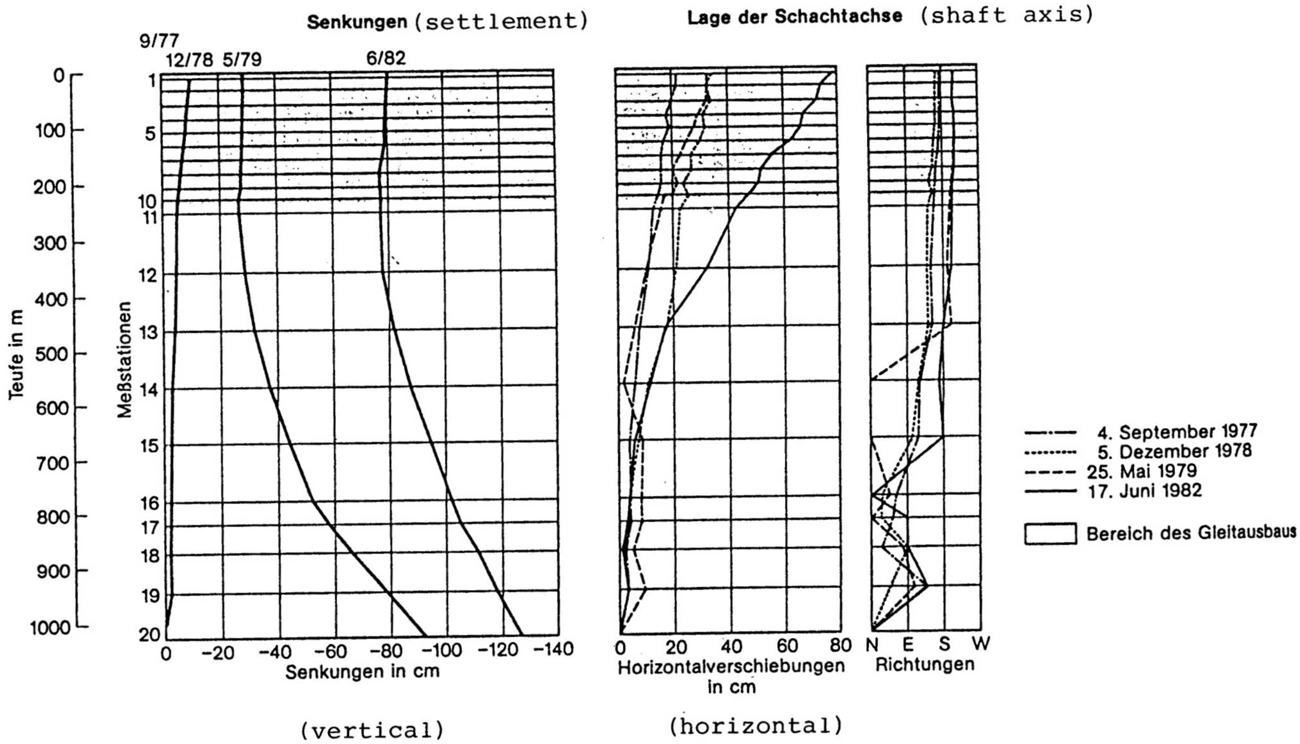

Figure 14. Working beneath Auguste Victoria no.8 shaft; settlement and changes in position of shaft axis

3. KAMPSCHULTE, R.M.; LEHMANN, W.; LINK, H. (1964): "Das Abteufen und Ausbauen der Gefrierschächte Wulfen 1 und 2. Glückauf 100 (1964) Nr. 25, S. 1473–1495
4. LINK, H. (1965): "Die Beanspruchungen eines Schachtausbaus aus Verschweißten Stahlblechen mit Zwischenbeton und Asphaltgleitfuge." Glückauf-Forschungshefte 26 (1965) Nr. 6, S. 341–355
5. LÜTGENDORF, H.O. (1967): "Der aus. serdichte gleitend Stahlbetonringaus-bau des Gefrierschachtes Auguste Victoria 8." Glückauf 103 (1967) Nr. 12, S. 553–560
6. BITTNER, F. (1985): "Die fertigstellung des Gefrierschachtes Voerde." Glückauf 121 (1985) Nr.19, S. 1423–1428
7. KLEIN, J.; RIEβ, H.G.; RITTER, H. (1987): "Abteufen und Ausbauen des Gefrierschachtes Sophia Jacoba 8." Glückauf 123 (1987) Nr.22, S.1395–1406
8. LINK, H.; LÜTGENDORF, H.O.; STOβ,K. (1985): "Richtlinien zur Berechnung von Schachtauskleidungen in nicht standfestem Gebirge." Herausgegeben steinkohlenbergbaverein, 3. Auflage 1985, Glückauf-Verlag, Essen
9. FALTER, B.; KLEIN, J. (1988): "Grenzlasten von Linern im Schachtbau." Bauingenieur 63 (1988) (to be published)
10. VOOYS, G.J.de (1955): "Das Bohrschachtverfahren. Bedingungen an den Ausbau eines Bohrschachtes beim schachtnahen Abbau. Vorgetragen vor dem 'Deutschen Ausschuβ für Schachtbau und Tiefbohrtechnik' beim Steinkohlenbergbauverein am 2. September 1955 in Essen
11. BECKMANN, D; GÄRTNER, D.; KLEIN, J. (1986): "Betrachtungen zur Spannungsund Stabilitätsberechnung von gleitendem Schachtausbau." Glückauf-For-schungshefte 47 (1986) Nr.4, S.163–170
12. LINK, H. (1967): "Entwicklung und gegenwärtiger Stand der Berechnung von Schachtauskleidungen in lockerem, wasserführendem Gebirge. Glückauf-Forschungshefte 28 (1967) Nr.1, S. 11–25
13. LÜTGENDORF, H.O. (1986): "Rückblick auf 30 Jahre gleitenden Schachtausbau." Glückauf 122 (1986) Nr. 17, S. 1101–1262
14. KLEIN, J. (1982): "Present state of freeze shaft design in mining." Symposium on Strata Mechanics, 5–7. April, Newcastle upon Tyne, UK; see Elsevier, Developments in Geotechnical Engineering Vol.32, pp. 147–153
15. AULD, F.A. (1985): "Freeze wall strength and stability design problems in deep shaft sinking—is current theory realistic?" 4. International Symposium on Ground Freezing, 5–7. August, Sapporo, Japan, pp. 343–349
16. WÖLFER, K.H. (1985): "Einfluβ der Teufsohle und des tragfähigen Auβen-ausbaus auf Stoßschiebungen in Gefrierschächten. "Glückauf-For-schungshefte 46 (1985) S. 150–158
17. BORM, G. (1985): "Wechselwirkung von Gebirgskriechen und Gebirgsdruckzunahme am Schachtausbau." Felsbau 3, Nr.3, S.153–158
18 . GILL, D.E.; LADANYI, B. (1987): "Time-dependent ground response curves for tunnel lining design." 6. International Congress on Rock Mechanics, Montréal, Canada; see Balkema, Volume 2, pp. 917–921
19. RIES, A. (1981): "Geschichtliche und technische Entwicklung des Gefrierverfahrens im Schachtbau." Schacht- und Tunnelbaukolloquium, 12. Juni, Düsseldorf erschienen in Glück-auf 118 (1982) Nr.2, S. 2–12

20. JESSBERGER, H.L.; HEGEMANN, J. (1985): “Der tiefe Bergwerksschacht Voerde als Beispiel für den modernen Gefrierschacht”. Vortrag auf der STUVA-Tagung ’85 in Hannover; erschienen in Forschung und Praxis der Studiengesellschaft für unterirdische Verkehrsanlagen e.V.-STUVA, Köln (1986) Nr. 30, S. 38–46
21. LÜTGENDORF, H.0. (1986): “Die Konstruktionsprinzipien des gleitenden Schachtausbaus für Gefrierschächte”. Glückauf 122 (1986) Nr. 19, S. 1258– 1262
22. RITTER, H. (1985): “Erfahrungen und Anforderungen des Landesoberbergamts Nordrhein-Westfalen.” S. 111–122 in: “Handbuch des Gefrierschachtbaus im Bergbau.” (siehe Literaturstelle 1.)

Excavation, support and lining of the LEP, Point 8, machine shaft

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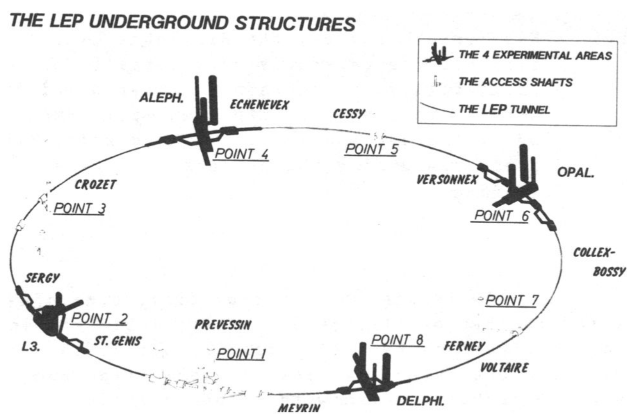

Fi gure 1 Layout of the LEP machine complex

SYNOPSIS

CERN, the European Laboratory for Particle Physics, is at present constructing a Large Electron Positron (L.E.P.) colliding beam machine adjacent to existing laboratory facilities, sited astride the French-Swiss border, just outside Geneva.

The LEP Machine is to be installed in a 27 kilometre long, quasi-circular ar tunnel at depths varying between 50 and 170 m below ground-leve1. Around the circumference are sited four experimental caverns, in which detection equipment will be mounted and aligned to observe the behaviour of particles created by the collision of circulating electrons and positrons, (see figure 1).

Access to the LEP machine interaction zones and tunnel is provided by a set of 20 verti cal shafts. This paper describes the various tech niques employed to carry out the excavation, temporary support and lining of the PM 85 shaft. The shaft, which has a finished diameter of 9.1 m and is 90 m deep, provides personnel, plant and services access to the LEP machine level at the Point 8 interaction zone, (see figure 2). The sink was performed through a series of water bearing glacial moraines into the bedrock, in which the main tunnel and cavern complex are constructed.

To minimise the start-up time of the underground Civil Engineering works, notably the installation of Tunnel Boring Machines (T.B.M'.s), a combination of diaphragm walling and freezing was adopted to traverse the glacial moraines, while conventional drill and blast techniques were used in the bedrock. A final slipform lining was placed on completion of the associated underground works to render the shaft watertight and ensure long-term stability.

GEOLOGY

The major part of the LEP underground works is sited in the Leman Basin, an anticlinal depression, which is situated between the Alpine and Jura massifs, lying to the SE and NW respectively. For a limited distance, 2.5 km, the tunnel passes into the limestone strata of the Jura, elsewhere the tunnel and cavern excavation was performed in the Plain bedrock, the Molasse, a series of fresh-water perideltaic deposits of the Tertiary (Oligocene) period. The Molasse, an alternating, subhorizontal series of sandstones and marls, was selected as the preferred excav-ation media for the tunnel as it had proved particularly amenable to excavation by mechanical means (TBM and Roadheader).

Towards the end of the Tertiary era, the molasse deposits were subject to a considerable amount of glacial erosion, this gave rise to the development of trenching in the bedrock surface, normally aligned in a NE-SW direction. Point 8 is situated on the western slope of one of these glacial trenches. During the Quaternary period, the basal area of the Leman Plain was

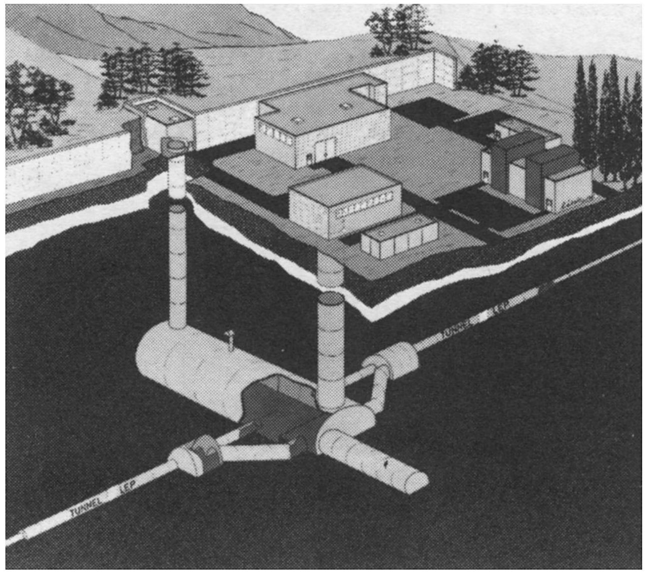

Figure 2 Schematic represantation of the Experimental Area surface and underground structures, showing the PM Shaft in the foreground

overlain by glaciation of the Rissienne and Wurmienne periods. At Point 8 the moraine deposits give an overburden cover of approximately 70 m.

Quarternary Moraine Deposits

Owing to their glacial origin the moraine deposits are neither consistent with depth nor horizon, however site investigation did allow a general, simplified section for the region to be established, as shown in figure 3.

The individual moraine formations can be described as follows,

I. Fluvio-glacial deposits; these form the plain sub-soils in the LEP area, where they are relatively shallow. The deposits consist of sandy gravels with numerous stones and boulders of Alpine origin. The formations have very 1ow cohesion, (see figure 4), and are relatively permeable.

II.a. Wurmienne silt Moraines; the Wurmienne deposits form the majority of the depth of the glacial overburden. The moraine varies from horizon to horizon, the fine—nd coarse-grained content fluctuating considerably. A silt and silt-clay phase exist, both are compact and of low permeability containing a large number of Alpine stones. Intraformational lenses of gravels and sands also exist.

II.b. Deep Stoney Wurmienne Moraines; these deposits mainly consist of compact sandy gravel beds of low cohesion. The formation is, in general, rich in Alpine stones and boulders, and locally, cemented conglomerates are present. This formation has a high permeability and acts as the reservoir formation for regional water supplies.

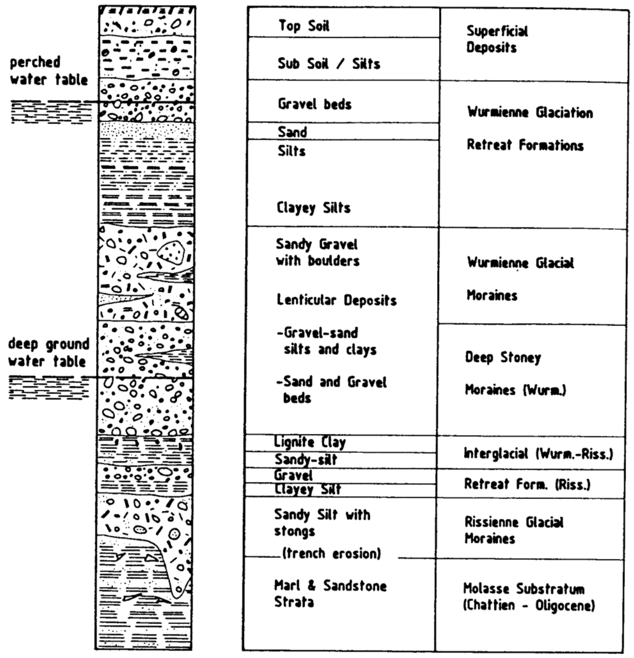

Figure 3 Typical cross-section through the glacial moraine deposits of the Leman Basin

Fig. 4—Soil characteristics of the Point 8 Moraine deposits

Soil Type	Deformation Modulus of	Angle, degrees Internal Friction	Undrained Cohesion kPa	California Bearing Ratio %
Fluvio-glacial deposits	50–80	35–38	0.5	>25
Wurmienne silt moraines	40–80	31–34	15–25	>20
Wurmienne silt-clay	40–70	29–31	15–25	>20
Wurmienne deep stoney stoney moraines	100–30	38–40	0–20*	>50
Rissienne silt moraines	50–100	31–34	20–30	>20

(* varied as a function of cementation)

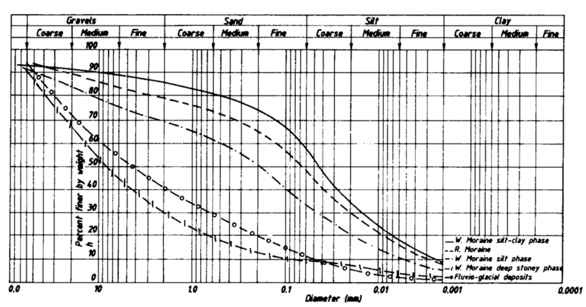

Fig. 5—PARTICLE SIZE DISTRIBUTION CURVES FOR THE POINT 8 MORAINE DEPOSITS

III. Rissienne Moraines; the Rissienne clay-silt moraines are generally very compact, of 1ow permeability and contain a number of Alpine stones. This deposit had been well compacted by the application of high consolidation pressures during the periods of glacial loading.

As is seen from the particle size distribution curves (figure 5) and the table showing the moraines’ mechanical characteristics, the set of media presented a variety of excavation and support problems; even within the individual formations considerable changes in the matrix composition occurred. To ensure ground stability and inhibit water inflow, systematic wall support was installed prior to excavation.

Molasse

The Molasse, which forms the Plain bedrock, consists of an irregular, alternating, sub-horizontal series of marl and sandstone deposits varying in thickness from 0.1 to 2.0 m. The Uniaxial Compressive Strengths (UCS) normally lie in the range of 8 to 20 MPa. However, certain of the marl strata were considerably weaker (UCS down to 1–2 MPa). These weak marl strata were particularly susceptible to hydration and swelling which could create short and long-term stability problems and the rapid application of a shotcrete lining was required.

Site Hydrology

At Point 8 two types of water-table are encountered in the glacial moraines. Local superficial or perched water-tables are present in the fluvio-glacial silt-gravel lenses; these are normally supplied directly from the surface and are contained in matrices of relatively 1ow permeability. The second, a deep groundwater-table, present in the basal moraines of the Wurmienne deposits, is used as a source of drinking water in the region and is mainly supplied directly from the Jura massif.

Given the presence of horizontal, semi-impermeable, marl bedding in the Molasse the set of bedrock strata can be considered to be unsaturated and quasi-impermeable.

For the LEP shafts it was most important to limit the water inflow into the underground works to a minimum both during construction and operation of the machine. Environmentally pollution or draw-down of the water-table was unacceptable, the deep water-table being used for public consumption. This dictated the use of non-pollutant support techniques, which did not require embarking up on extensive injection campaigns. For the underground works in the Molasse, the presence of water could provoke superficial instability on the excavation surface or penetrate into rock fractures, thus reducing internal cohesion and internal friction factors. During the life of the machine the presence of water would have had a detrimental effect upon the operational efficiency of the numerous electrical-electronic components present in the laboratory facilities.

EXCAVATION AND TEMPORARY SUPPORT

As can be seen from the vertical cross-section of the PM 85 shaft, shown in figure 6, the molasse bedrock is overlain by a relatively thick layer of Rissienne moraine; this lies at depth (over 60 m) under the less compact Wurmienne deposits. Once anchorage had been obtained into this deep compact moraine, excavation was carried out by backactor, support being given by shuttered concrete placed in 3 m lifts directly upon completion of the excavation. In the Molasse, explosives were used and support given by reinforced shotcrete.

Above the Rissienne horizon, support of the sidewalls was necessary prior to excavation, owing to the varying nature of the moraines and the presence of the superficial and deep water tables.

To give temporary support to the LEP shafts where deep glacial moraines were present a combination of contiguous diaphragm walling and freezing was used. The diaphragm walling was 1imited to a depth of 40 m in the moraines; below this depth the panel thickness, necessary to compensate for individual panel deviation and inter-panel misalignment but still guarantee a continuity of the circular structure, became excessive. For depths exceeding 40 m in the moraines, support around the shaft excavation was provided by the use of freezing techniques for the full depth of the shaft through superficial and deep water-tables into the impermeable substratum. However, at Point 8, owing to the tight time schedule, it was decided to combine the use of diaphragm walling and freezing. This was possible owing to the presence of a continuous, impermeab1e horizon situated at a depth of approximate1y 27 m, which acted as a “cut-off” for the superficial water-tables. Here, to minimise the prolonged start-up time required to freeze the upper water-tables, the moraines were supported by the use of a polygonal diaphragm wall. The drilling of the freeze-holes into the Rissienne moraine anchorage was carried out at the same time as the walling operations and the freezing process commenced during excavation of the upper section of the shaft, thus permitting closure and development of an ice curtain prior to excavation, at depth, below the deep water-table.

The PM 85 shaft was considered to be of crucial importance to the overall success of the LEP project. The two Tunnel Boring Machines, as originally proposed for the excavation of over twenty kilometres of the circular main tunnel, were both to be assembled and start their drives in clockwise and anticlockwise direction from Point 8 to tunnel towards Point 3 and Point 4 respectively (see figure 1). Early access to the LEP tunnel level at Point 8, via the PM 85 shaft, was thus essential and its excavation lay on the critical path of the construction schedule. The photo in figure 7 shows the arrival of one of the TBMs at the LEP machine tunnel level at the start of the underground assembly.

Figure 8 shows the actual advance rates achieved during the support and excavation of the PM 85 shaft. The overlap of diaphragm walling and freeze tube drilling activities was originally intended to allow excavation to continue uninterrupted within and below the diaphragm walling through the unsaturated moraines into the, by then, frozen deep water-table. The proposed schedule was not met due to strikes by the Contractor’s personnel which effectively doubled the overall sinking time of the shaft. As can be seen from the works schedule subcontractor operations were not affected by the strike.

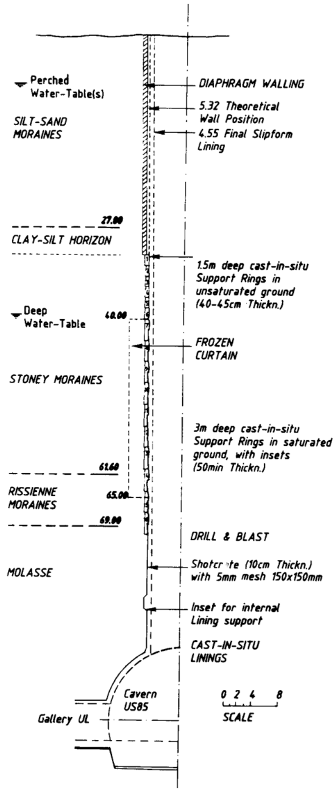

Fig. 6-CROSS-SECTION THROUGH THE PM85 SHAFT DESCRIBING THE TEMPORARY SUPPORT MECHANISMS EMPLOYED

Diaphragm walling

To provide temporary support for the upper section of the PM 85 Shaft, through the superficial water-table, down to −25 m, a polygonal, contiguous diaphragm wall was executed. The diaphragm wall, of 0.62 m thickness, was designed to support the saturated sand-silt lenses present in the upper moraines and was anchored into the impermeable silt-clay horizon. This horizon was encountered at greater depth than anticipated, between 26 and 29 metres below surface. Excavation of the individual panels was performed by rope supported panel bucket. The plant originally used for the panel excavation, a jawed bucket, rigidly mounted on to a telescopic kelly mast, proved too susceptible to deviation. Boulders, which occurred in the moraine matrices, were passed either by the use of trépanning or by normal bucket. Below 25 m, the design depth of the cylindrical structure, deviation of the individual panels was, in some cases, excessive (more than 30 cm). Additional wall reinforcement from the −27 m level was necessary to ensure stability of the base of the cylinder. A circular ring of shuttered concrete was placed to compensate for the ovalisation which occurred at the base of the diaphragm walling.

Generally, overbreak outside the panel profile was limited, even in the boulder rich beds of low cohesion. The panel excavations, around the PM shaft did however prove susceptible to overbreak in the sand-silt deposits, notably on the inside wall of the structure. This phenomena was thought to be induced by fluctuations in the level of the internal perched water-table. A relief hole was drilled, down to the water-table, within the cylindrical structure and a pump installed to maintain the water-table at a level well below surface. The drainage of this water-table considerably reduced the overbreak volume in the sand-silt horizons. A global overbreak value of 16% (given as a percentage of the theoretical wall volume) was calculated, from concrete consumption figures, at Point 8. Given the relative stability of the openings, panels were often poured simultaneously. The panels were individually reinforced to ensure their vertical stability; horizontally, continuity of the circular profiles was necessary to ensure the continuity of transmission of the compressive stresses. The “tremie” system was used for concreting, a strict control being maintained on the concrete, bentonite and tube joint extraction operations. At Point 8, where two diaphragm wall shafts, of similar dimension, were excavated a walled surface of over 2200 m2 was created at an average rate of 3 m/h within a five week period.

Freezing

To carry out the freezing of a cylinder of ground around the PM 85 shaft, at depth, an original circular series of 36 holes was drilled at 1.2 m centres from surface down into the Rissienne moraine. An anchorage of 5 m was given into the Rissienne’, which was of negligible permeability (1×10−8 m/s), and of high cohesion. Given the variable nature of the glacial moraines,

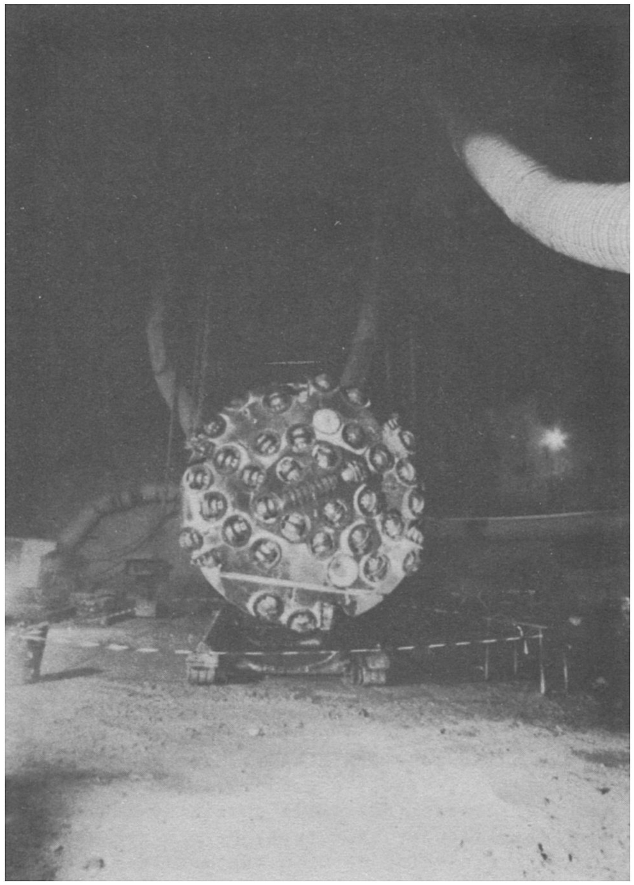

Figure 7 View showing the arrival of a double shield TBM at the PM Shaft base

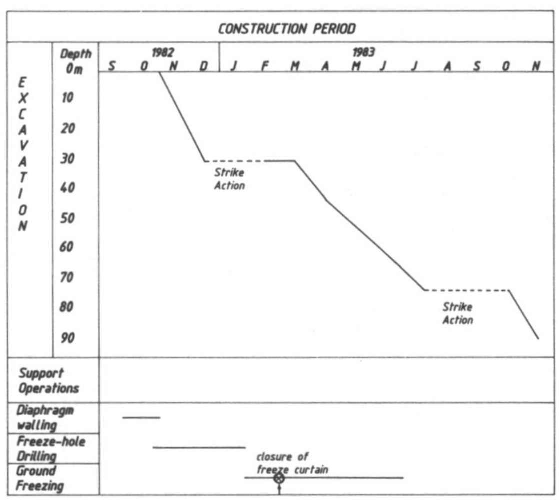

Fig. 8-PM85 EXCAVATION AND TEMPORARY SUPPORT WORK SCHEDULE

notably the extent of consolidation and granulometry make-up, the down-the-hole Odex drilling system was adopted, the drill casing being driven upon advance. Once drilled to full depth, the drill string was removed and the closed-circuit plastic freeze tubing was installed. A bentonite slurry was injected to reduce local ground water movement around the freezing zone. This latter operation minimised the migration of water during the heat exchange process, hence reducing the time for the onset of freezing.

After drilling the original set of freeze holes, a down-the-ho1e survey was carried out to check that no "window" existed in the potential freeze curtain. As a result of deviation of the freeze holes at depth, auxiliary freeze tubes were necessary to ensure that the maximum horizontal inter-hole distance did not exceed 2.1 m. This maximum inter-tube distance was fixed to

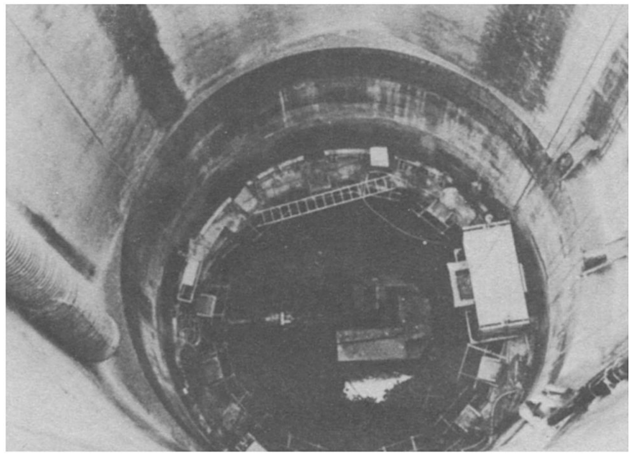

Figure 9 View showing backactor excavation and ring shutter installations below the diaphragm wall transition level

ensure that the formation of the ice wall was obtained evenly around the shaft perimeter before the planned start of the excavation operations.

Once the freezing process was commenced by the closed circulation of a brine (Ca Cl) solution at temperatures between −30 and −25 deg. C., the propagation and evolution of the process was observed by temperature and piezometric measures, performed in auxiliary bore-holes, inside and outside the ice wall perimeter. Confirmation of the ice wall closure was obtained when a differencȩ was noted between the internal and external water-table levels. Once closed, the progression of the ice-wall, and consequent expansion, generates an upward movement of the trapped water-table.

The curtain closure time for the deep water-table was calculated to within 10% (46 days compared to 42 days anticipated) and maintained for a period of 125 days to permit excavation and support in this zone to be carried out. Originally a more rapid rate of excavation had been expected but owing to difficulties encountered by the main contractor, including a strike by his personnel, the maintenance period was prolonged. This in turn led to the progression of the ice-wall well into the shaft excavation zone.

As could be expected, the heat exchange process was concentrated in the deep water-table. However, ground freezing occurred for the whole depth of the shaft, particularly in the perched water-tables zones and, to a lesser extent, in the unsaturated moraines. Behind the diaphragm walling this gave rise to horizontal panel fracturing and a heave of the order of 0. 5 cm was noted at the shaft collar.

In the final four metres of the diaphragm walling, where excessive interpanel deviation had been found and through the moraines, at depth, cast-in-situ concrete rings were poured in passes of 1.5 and 3.0 metres. The "Moroccan walliing" provided temporary support in the moraines, above, through and below the deep water-table, into the impermeab1e Rissienne Moraine, during and after the freezing operation up until placement of the slipform lining.

The temporary lining shutter, as seen in figure 9, remained suspended in the shaft during excavation of the next pass and was then positioned for placement of the concrete pour. As may be appreciated, the joint contact between consecutive passes was not executed in the best of conditions and a grouting campaign concentrated below the deep water-table was carried out, once the passage through the stoney moraines had been completed and the surrounding ground defrosted. Cement and chemical-based grouts were injected locally at the construction joints to reduce to a minimum the water inflow occurring, prior to continuing the excavation into the bedrock strata.

Upon arrival in the sandstone and marl strata of the molasse bedrock/drilling and explosives were used to break out the central bulk of the rock; excavation to the shaft wall was achieved by hydraulic hammer to minimise blast damage to the surrounding rock. The variable strength and horizontal bedding of the Molasse did not facilitate explosive breakage, however 3 m pulls were achieved during sinking of the Machine shafts. At Point 8, owing to the proximity of the cast-in-situ rings and cavern wall, 3 m lifts were not attempted.

Upon formation of the shaft wall, wire mesh and shotcrete were applied. Rockbolting was installed systematically at the base of the shaft to support the cavern intersection zone. In all cases a rapid application of an initial shotcrete layer was necessary to protect the Molasse marls against alteration.

Final Lining

As can be seen from the vertical section figure 6, insets were placed in the Moroccan walling temporary support lining, from −40 m down. These insets were placed to support the weight of the final slipform lining and limit the loading of the shaft-

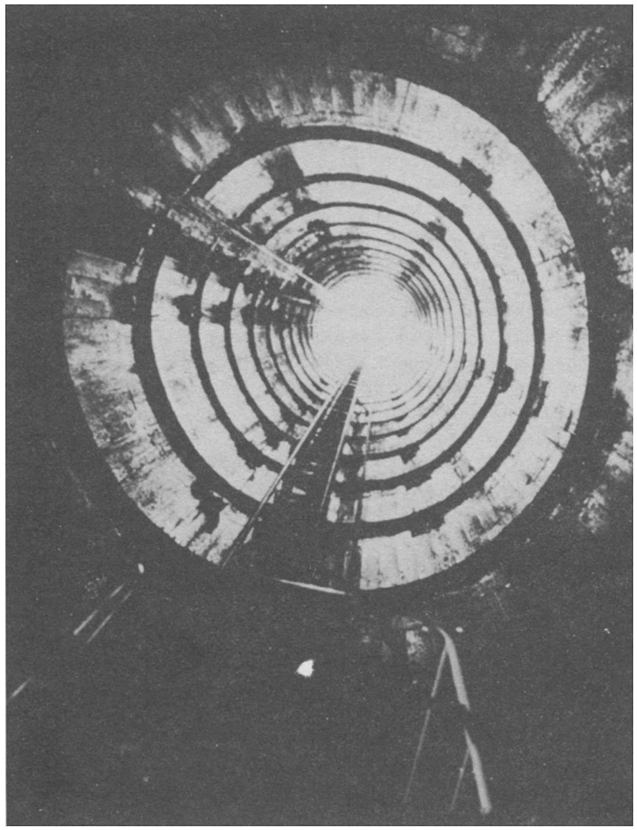

Figure 10 View looking up the temporarily supported PM 85 Shaft, showing the slip-form insets

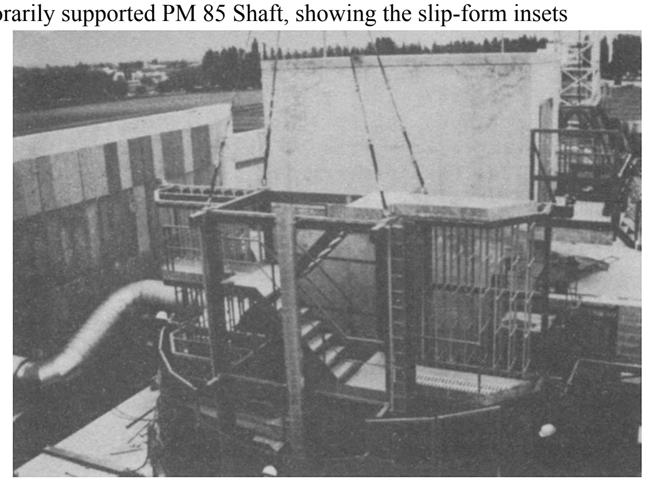

Figure 11 Topping-out of the PM prefabricated units prior to services (cable, pipe etc…) installation

cavern lining in the intersection zone. This was achieved by placing a shutter negative in the cast-in-situ rings and by making enlargements in the bedrock excavation.

The temporary lining was designed to give support throughout the duration of the underground excavation and lining operations. To ensure the long-term stability and watertightness of the shaft, a waterproof membrane and slipform lining were placed upon completion of the Point 8 tunnel and cavern works. The waterproofing and concrete operation were performed simultaneously using a three-level galloway stage, with the waterproof complex being placed on the upper level prior to the pouring of the slipform. The slip-form shutter, of 1 m depth achieved 24 hr lifting rates of between 5 and 8 I. m.

To reduce the water inflow and hence limit pumping at the Point 8 sump, a cut-off was placed behind the in-situ lining in the impermeable bedrock. This was achieved by sealing the waterproof PVC membrane onto a prepared contact surface, a non-shrinkage mortar which had previously been poured into a bedrock inset. This cut off restricts the passage of water through the zone and limits the water-table draw-down.

CONCLUSIONS

The combination of ground support mechanisms successfully supported the shaft during the excavation period up until placement of the final slipform lining. The use of freezing techniques allowed the excavation to be undertaken in good conditions, eliminating water inflow and giving rigid ground support. Excavation of the frozen moraines below the water-table often proved very difficult as propagation of the ice-wall, well into the excavation profile, rendered the matrix very hard for break-out using hydraulic hammer. The use of explosives was limited owing to the close proximity of the plastic freeze tubes which were brittle at operating temperatures of down to −30 degrees C

Finally, it is worth noting that despite the best laid plans of the design engineers it is often the on-site elements which control the overall success of a project. However, despite the delays encountered resulting from strike action, the Plain sector is now complete and the physicists are preparing to commence their experiments at all four LEP interaction zones by mid-1989.

1. Amberger G.-La molasse du bassin genevois. Thema: La recherche scientifi que dans les hautes écoles et universités suisses 1986. p. 5–8.

Dieu F., Laughton C.—Shaft sinking in difficult ground conditions for the LEP Project, Geneva, Switzerland In Proceeding of the Rapid Excavation and Tunnelling Conference, New Orleans— June 1987ch. 57– p. 924–937.

Gonze P., David E.—Le traitement du sol par congelation pour le fonçage de cing puits d'accès au LEP. Travaux—June 1988—p. 34–37.

Hotellier J.F., Rebuffe P.—Les parois moulées dans la construction du LEP. Travaux—June 1988-p. 31–33.

Laporte H.—Le LEP, grand projet souterrain. Tunnels et ouvrages souterrains No 63, May-June 1984-p. 115–122.

Sinking of the Asfordby mine shafts

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SYNOPSIS

This paper reviews the techniques involved in the construction of two 7.32 metre finished diameter shafts through the heavily water bearing Bunter Sandstone formation encountered at the Asfordby Mine.

The operation entailed a sub-surface ground freezing exercise starting from a level 270 metres below surface and extending to 405 metres depth.

INTRODUCTION

British Coal is investing in the new Asfordby Mine, near Melton Mowbray in the N.E.Leicestershire coalfield.

The mine will be the largest in the Midlands with a planned output in excess of 3 million tonnes/year of low cost coal for supply to nearby power stations.

Twin vertical shafts, one to carry men and materials and one for skip winding of coal, have been sunk to gain access to 146 million tonnes of reserves in the Deep Main, Parkgate and Black-shale schemes.

Work on site commenced in 1985 and will take approximately 10 years to achieve maximum production levels. A specialist contractor is employed for the shaft sinking, ground freezing and pit bottom development works.

GEOLOGY

To prove the geology and quality of the coal, some 25 deep exploration boreholes were drilled around the area between 1974 and 1983. The geological section of the shaft site is reproduced in Figure 1.

The presence of the Keuper Waterstones and in particular the Bunter (Sherwood) Sandstone were proven to be heavily water bearing. Pumping tests carried out indicated that the possible water make into an unlined shaft, without prior treatment, would be approximately 76 litres per second (1000 gpm) under virtually full hydrostatic head.

This aquifer was identified some 330 metres below surface extending to a depth of approx. 390 metres. This section is overlain by the Keuper Marls and Lower Lias, and is situated above the Westphalian Coal Measures, all predominantly dry formations.

In addition an igneous intrusion was identified that could possibly encroach on the lower portion of the Upcast Shaft Sump. This intrusion was located by boreholes in the vicinity of the Upcast Shaft and was known to have fissures containing water at up to 70 bars (1000 psi). This intrusion was therefore also a potential aquifer.

CONSTRUCTION

Ground Treatment

Several methods of ground treatment were considered by British Coal to enable the shafts to be sunk efficiently and safely. The final decision was made in favour of Ground Freezing for the Bunter Sandstone.

It was decided to carry out a sub-surface freeze from an underground chamber constructed above the aquifer. The chambers were constructed at a depth of approx. 275 metres below surface which provided a minimum 45 metres of cover to any known waterbearing zone. The freeze cover was extended into the Coal Measures and terminated at a depth of 405 metres to form an impervious cut off. This would prevent any possibility of ground water permeating under the ice wall and into the excavation.

The Igneous intrusion in the Upcast Shaft was to be investigated by cover drilling from the sump floor some 20 to 24 metres above the potential aquifer and if located and found to be waterbearing treated by cement and or chemical injection.

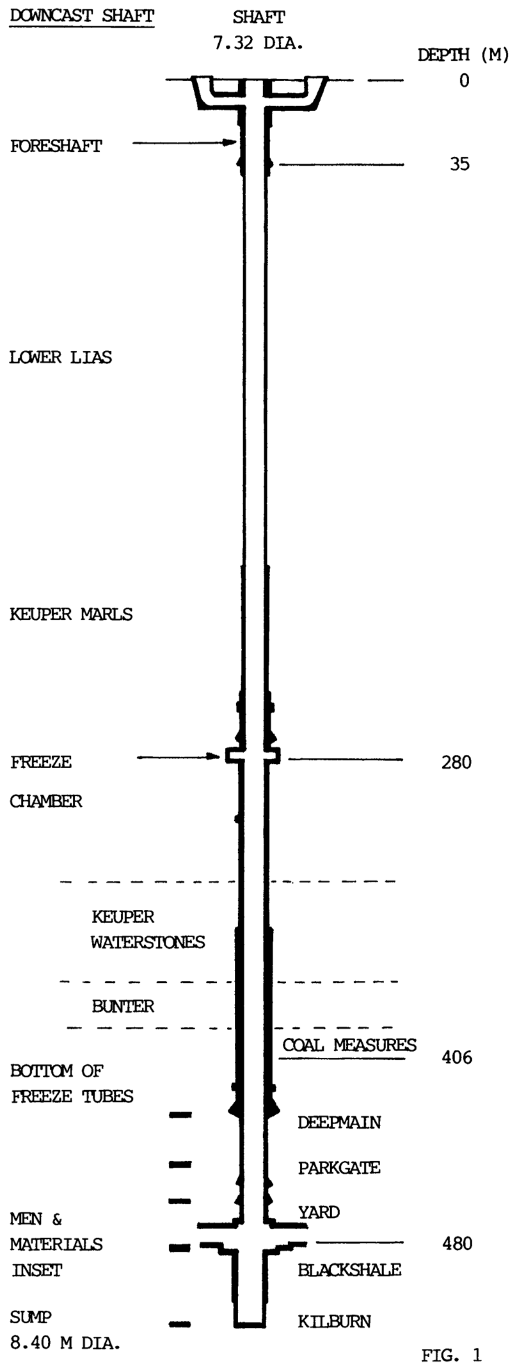


FIG. 1

Construction Sequence

The sequence of operations for the shaft sinking contract was as follows;

Upcast Shaft	Downcast Shaft
Freeze hole drilling	
Permanent headgear, foundations, foreshafts and fan drift	Freeze hole drilling
Shaft sinking to freeze chamber	Permanent headgear foundations. Foreshafts and Air Inlets

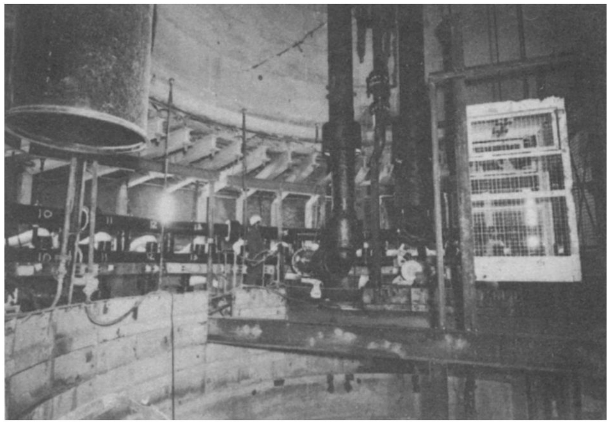


FIG. 2

Upcast Shaft	Downcast Shaft
Freeze Chamber Construction	Shaft sinking to freeze chamber
Freeze/permanent head-gear—Phase 1	Freeze Chamber construction
Sink to Inset	Freeze/permanent head-gear—Phase I
	Sink to Inset
Inset construction—Phase I	
	Inset construction—Phase I
Initial Development	
Inset construction—Phase II	Inset construction—Phase II
Sump—Phase I	Initial Development
Pit bottom drivage	Sump Shaft furnish/commission
Sump—Phase II	Pit Bottom Drivage
Shaft furnish and commission	

Although the Downcast Shaft started twenty weeks behind the Upcast Shaft, by the time both insets were completed the time lag had reduced to two weeks. This was partially due to the additional pit bottom development done from the Upcast Shaft, however, there is no doubt that the lessons learnt in the Upcast Shaft assisted the progress of the Downcast Shaft.

During the period of freezing the Bunter Sandstone in both shafts the opportunity was taken to construct the first phase of the permanent concrete head towers. These were constructed to a level above the sinking headgears during the time that the freeze was taking place and the shaft sinking winders were decommissioned. During this period access was maintained to the mid shaft freeze chamber using small single drum “Mary Anne” hoists operating through the Upcast Fan Drift and the Downcast Air Inlet. (Figure 2 shows the Downcast Shaft Freeze Chamber with “Mary Anne” cage landing level).

Thus, once the ice wall had achieved the required thickness and the sinking headgear recommissioned and braced against the permanent tower, sinking could recommence concurrent with the completion of head tower construction and installation of the permanent tower mounted winders.

FREEZE HOLE DRILLING

Mobilisation of drilling equipment to site commenced on 5 August 1985.

Three drilling rigs were brought to site, two Failing 2000’s and one Failing 2500 and all systems were commissioned ready for drilling on 21 August.

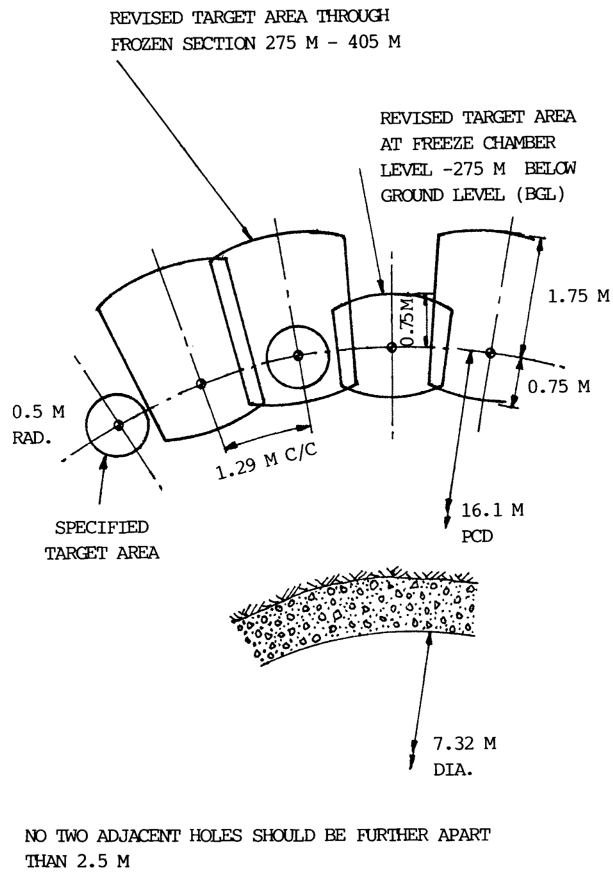


FIG. 3 DRILLING TARGET AREA

Each shaft was to be surrounded by 39 freeze holes spaced at 1.29 metre centres around a 16.1 metre diameter circle. In addition, 3 monitoring holes were to be drilled per shaft at positions outside the freeze circle.

Drilling commenced at 305 mm diameter through the surface deposits to a depth of approx. 50 metres. A steel conductor casing 220 mm diameter was then lowered in and cemented into position. This casing prevented erosion of the soft surface deposits and interconnection between adjacent holes being drilled.

Drilling then recommenced at 200 mm diameter.

A drilling target area of 0.5 metres radius around each hole's planned position was initially specified.

However, initial drilling operations on site showed that substantial difficulty was being experienced in maintaining the hole within the target. An enlargement of the target zone was agreed with the Client after some 8.5 weeks drilling. The revised target zone, (Figure 3) had a similar maximum hole spacing but allowed for more deviation to occur, especially below the freeze chamber level. This additional target area was used to carry out necessary corrections to the direction of the hole when a survey indicated that it was deviating towards the target limit.

Freeze Tube Installation

Upon completion of drilling each hole to 405 metres depth and verification of the survey results, the hole was conditioned with clean mud and the drill string removed. A cementing string was then lowered to the hole bottom and a predetermined quantity of Class B oilwell cement was injected. The cement was mixed to very accurate proportions with a slurry weight of 1. 68 kg/litre. Fluid loss additives and retarders were included in the slurry to give a 24 hour working period before the initial set. The cementing string was then withdrawn and the 125 mm O.D. freeze casing was assembled, each joint being torqued up to specific limits and externally pressure tested to 18.62 MPa (2700 psi).

The freeze casing, approximately 130 metres long, had a sealed top and bottom plug arrangement. A steel inner pipe 65 mm dia. was also installed at this time and the casing string filled with water to provide negative bouyancy.

When this arrangement was completed, a detaching mechanism was connected to the top plug and the whole casing string was gently lowered into the cement. When the casing was at the correct horizon, important for later operations, the casing was detached from the lowering string. The lowering string was fitted with a circulation port at the bottom and, after confirmation that the casing was detached, mud was slowly circulated and the surface returns monitored to ensure that cement was noted. This then confirmed that the freeze casing was totally cemented and no possibility existed of water being able to find its way up the drill hole annulus to flood the shaft during freeze chamber construction.

Immediately the above was confirmed, the hole above was then cemented to the desired level using a Class B cement slurry with accelerator as required.

Drilling and freeze hole abandonment works were completed on schedule at the Upcast Shaft on 26 January 1986 and 5.5 weeks ahead of schedule at the Downcast Shaft on 16 April 1986. The use of a fourth rig was necessary on the Upcast Shaft for a period of 6 weeks to achieve the programme due to the directional drilling difficulties experienced in the first 8.5 weeks of drilling. The total drilling carried out for the two shafts amounted to 34424 metres with the installation of 10220 metres of freeze tube casings.

Foreshafts

On completion of freeze hole drilling and before foreshaft excavation, large 2.1 metre diameter bored piles were constructed at both shafts to act as the foundation for the permanent head towers. These piles, 13 at the Upcast and 10 at the Downcast were bored approximately 40 metres deep through the surface deposits to form a solid foundation in the mudstones of the Lower Lias.

During initial pile boring it was found that a thin limestone band, some 12 metres below the surface appeared to contain a considerable amount of water.

After consideration of the effect this water might have on the foreshaft excavation it was decided to treat this limestone band with a simple, low pressure cement injection in order to reduce the potential inflow. 17 Holes were drilled at the Upcast in the area surrounding the foreshaft and fan drift and, after establishing 6 metre standpipes these hole were deepened into the limestone band to be treated. 35 Tonnes of cement was then injected into the band until a central test hole showed that the treatment had been successful and that a potential water make of some 115 litres per minute had been reduced to 2 litres per minute.

A similar exercise around the Downcast Fore-shaft and Air Inlets excavation involving 20 holes and 30 tonnes of cement had a similar result.

Foreshaft excavation was carried out using a Takeuchi TB45 mini excavator loading into kibbles which were hoisted out of the shaft by 10 tonne Scotch Derrick.

Temporary ground support in the Upcast Shaft was provided by sheet piles down to 12 metres depth and below that by rockbolts, mesh and shotcrete. Below the fan drift opening circular steel ring beams were installed to provide additional support.

In the Downcast Foreshaft, that being a simpler circular shaped excavation, the shaft itself was supported during excavation by a 150 mm thick sacrificial concrete lining cast behind a simple shutter in 1.5 metre lifts as excavation proceeded. The Air Inlet excavations were sheet piled.

Both methods of foreshaft temporary support proved successful and both excavations stood open without problems until completion of the permanent reinforced concrete lining. The foreshaft construction was complete ready for installation of shaft sinking equipment by 5 July 1986 in the Upcast Shaft and 4 October 1986 in the Downcast Shaft.

SHAFT SINKING AND LINING

Commissioning of Sinking Equipment

During foreshaft construction work was progressing at both shafts on the build up of the shaft sinking equipment. Each shaft was equipped with a double drum kibble winder (one drum clutched) a four drum stage hoist, a four deck stage and a headgear.

Whilst the winders were being installed in their permanent positions the stages and head-gears were built up as much as possible alongside the shaft so that they were available for lifting into position immediately the foreshaft was complete to bank level.

The sequence of installation was;

i) Lift stage into shaft and suspend.
ii) Install bank doors and decking.
iii) Lift/slide headgear into position and bolt down.
iv) Install pulley platform on headgear.

v) Rope up and hang stage and commission winders.
vi) Install muck chutes and commission.

The installation and commissioning operations took some 4 weeks at the Upcast Shaft and 5.5 weeks at the Downcast Shaft where access restricted the amount of pre-erection that could be done.

Shaft Sinking Equipment

The shafts were to be sunk using conventional drill and blast techniques.

Considerations that effected the choice of equipment were;

i) The relatively shallow depth of the shafts.
ii) The strata conditions.
iii) The capacity of the stage hoists.

Drilling

Although consideration was given to the use of shaft sinking drill jumbos these were rejected primarily because the soft nature of the strata.

The disadvantage of the jumbo is that it takes a relatively long time to set up for drilling and to remove after drilling. The main advantage is that, once set up, the drifters mounted on it are capable of faster penetration rates than hand held drilling machines.

In softer strata, mudstones, marls etc. as prevalent at Asfordby, hand held drilling machines can achieve relatively fast drilling rates and the main advantage of jumbos over hand held machines is diminished. The disadvantage still exists however.

Both Asfordby shafts were sunk using predominantly rotary percussive Holman Silver 70 jackhammers although in some of the weaker strata rotary Turmag drills were sufficient. The Silver 70's proved good strong machines and it was possible for an 8 man team to drill a 4 metre round of some 120 holes in less than 4 hours.

Blasting

Blasting was done with 80% gelegnite and half second delay detonators initiated using the Magnadet system. Although some initial problems were experienced with the complex Magnadet exploders, once overcome the system proved very successful and for shaft sinking Magnadets must certainly be considered an advantage over standard electrically initiated detonators.

Mucking

The 3 types of mucking system considered for removal of blasted rock were;

i) Cactus grab slung below the sinking stage.
ii) Cryderman grab—wall mounted.
iii) Eimco 630 rockershovel.

Cactus grabs and underdeck mucking units had been successfully used on the Selby shaft sinkings. However, the disadvantages of the system are that, a) it adds an extra 10 tonnes of weight to the stage and b) because the winch and traversing mechanism are slung below the bottom deck of the stage, the stage cannot be lowered down to the floor of the shaft and the concrete lining cannot easily be kept tight to the sump. This was considered to be a major disadvantage at Asfordby where two thirds of the shafts were to be sunk through mudstones and marls which, if wet, deteriorate very quickly. In these conditions it is prudent to have the ability to bring the permanent concrete lining as close to the advancing sump as possible, and for this reason the use of cactus grabs was discounted.

The Eimco 630 rockershovel has proven a useful and versatile tool for loading kibbles, particularly in larger diameter shafts and recent experience at Maltby had reinforced this view. This method of mucking was however initially rejected because it was felt that the 630, being a tracked machine that runs about on the muckpile, would soon bog down in the marls at Asfordby, especially if, as predicted, water was present in occasional limestone bands.

The initial choice was therefore made in favour of a mucking system consisting of 2 wall mounted "Herman" Cryderman grabs per shaft.

This system was put into use at the start of sinking of the Upcast Shaft and was used down to the freeze chamber level. It did not prove very successful however, mainly because of the difficulty in operating the units and the lack of skilled operators in the U.K.

Because of this and the fact that the strata below the foreshafts was proving dryer than anticipated it was decided to try the Eimco 630's in the Downcast. Although expensive on spares, these machnies worked successfully from the start as finding and training of operators was far less of a problem. The use of the Cryderman grabs was therefore discontinued and both shafts completed using Eimco 630's.

Lining

As sinking proceeded the permanent concrete lining was advanced in 6 metre lengths. Depending on ground conditions the lining was kept between 2 metres and 18 metres from the advancing sump.

Apart from a short section above each inset the lining was unreinforced mass concrete with a nominal thickness varying between 400 mm in the Coal Measures to 1050 mm in the lower section of the Bunter Sandstone aquifer.

The shaft concrete lining cycle consisted of;

The shaft concrete lining cycle consisted of;

i) Strip and lower kerb and 1st ring of shuttering.
ii) Line and level kerb on hanging rods (embedded in the concrete above).
iii) Fill kerb and "A" ring with concrete.
iv) Lower next four rings of shuttering, line and fill with concrete.
v) Lower last 3 rings of shuttering plus matcher and complete the concreting pour.
vi) Extend services.

Concrete was fed into the shaft down a 200 mm dia. pipe from surface, through a "dashpot", into the concrete distribution system on the stage and behind the shutters with 4 flexible 1500 mm hoses.

The maximum rate of concreting achieved was some 55 cu.m. per hour and the whole cycle, including extending services was achieved, on occasion in an 8 hour shift.

After overcoming initial teething problems with the systems a steady rate of 18 metres per week of shaft sunk and lined was achieved through the unfrozen sections of the shafts.

UNDERGROUND FREEZE CHAMBER

Construction

Construction of the freeze chamber began by driving a top heading in and opening out. Roof support was by radial beams supported at the shaft from hanging rods cast into the concrete lining and steel upright legs at the back of the chamber (Figure 4). Roof bolts and mesh provided supplementary support together with temporary square work. The freeze hole positions were located in the roof and as the chamber was further opened up, care was exercised during blasting to prevent damage to the freeze casings. Spoil was removed by Eimco 630's into the main hoist kibbles.

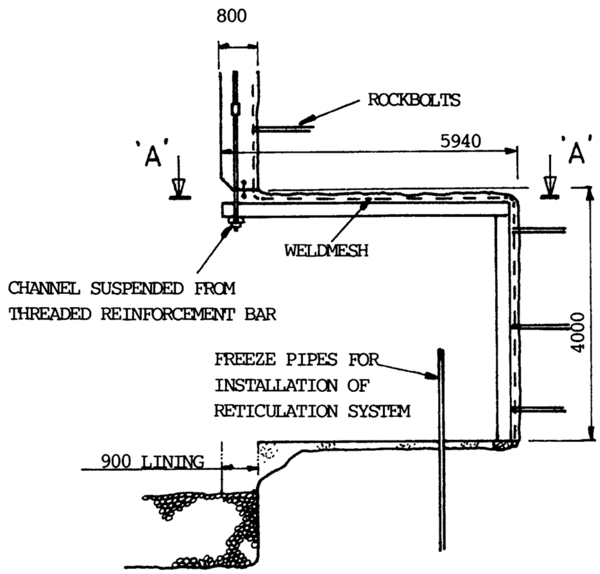
Exposed strata was also protected from long term weathering by the application of gunite.

Furnishing of Freeze Tubes

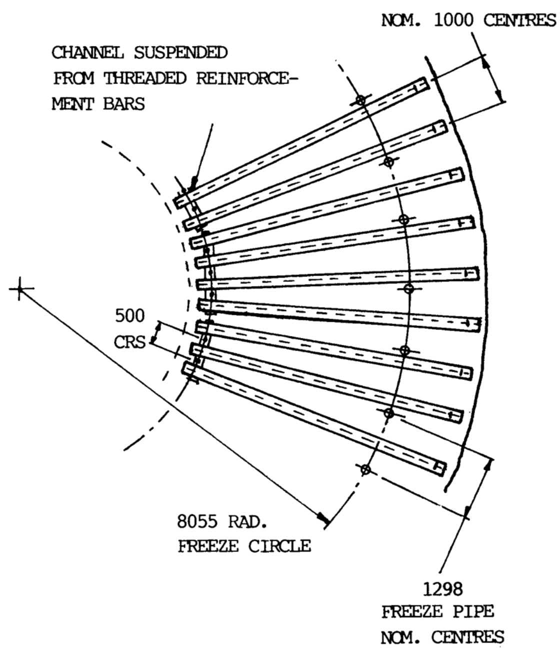
Once all the freeze casings had been located at the correct horizon and identified, the top plugs were removed to expose the male thread on the casing. Class 600 weld neck flanges which had been specially threaded were then screwed on and torqued up. An hydraulic pressure test to 1.5 times working pressure was then carried out for a 10 minute duration to confirm the competency of the joint and casing. The 65 mm dia. inner pipes were then connected onto the flow head arrangement and the flange faces torqued up.

The shafts were then sunk down to approx. 290 metres depth to accommodate the stage below the freeze chamber, and the backwall grouting operations were completed to surface.

Installation of the brine delivery and return ring mains was then carried out. These were suspended from the radial roof support girders of the chamber to eliminate any effect from heave which might occur in the floor both prior to and during freezing operations. Flexible connections between the ring mains and freeze tubes were made using high pressure, low temperature hydraulic hoses, incorporating a flow controlling valve.



FREEZE CHAMBER.



SECTION 'A-A'.

FIG. 4 FREEZE CHAMBER EXCAVATION AND SUPPORT

To protect the system from total loss of brine in the event of a burst hose, remotely operated main valves were fitted to the shaft range connections.

The freeze chamber delivery and return mains were then connected to the surface freeze plant by two 300 mm fully insulated shaft ranges. These ranges were supported by a stool 10 metres above the freeze chamber and held to the shaft wall by special sliding clamps. The shaft ranges were designated to accommodate a temperature range of +20C to −35C and vertical movement at surface of 165 mm.

GROUND FREEZING

Ice Wall Design and Formation

The ice wall, when formed, had to provide a structurally sound support for the prevailing strata conditions and methods of working.

A finite element model was analysed using data for Bunter Sandstone based on past experience. This indicated that the worst case would require an ice wall 1.5 metres thick. Allowing for the practical aspects and a factor of safety, the minimum desired ice wall thickness was set at 3.0 metres.

A finite element thermal analysis was also carried out using pessimistic data. The results indicated that the desired 3 metre thickness could be achieved within 15 weeks of freezing. In actual fact, the time to achieve the 3 metre thickness was 10 weeks for the Upcast Shaft and 12 weeks for the Downcast Shaft.

Freeze Plant

The installed freeze plant capacity was determined on the basis that both shafts may have to operate concurrently. Its total rated output of some 600 tonnes/refrigeration/day consisted of three screw type compressors and two reciprocating type with a combined input capacity of 2500 hp. Each compressor was mounted on its own self contained unit together with evaporator, condenser, control circuits etc. The primary refrigerant used was Ammonia with shell and tube or evaporative type condensers rejecting the heat to atmosphere.

Circulation of the brine to each shaft was carried out by a bank of four pumps arranged in parrallel. The flow rate for the freeze period was approximately 190 litres/sec. (2500 gals/ min).

Freezing Operations and Monitoring

During the freezing operations, the performance of the freezing systems and progress of the ice wall build up was constantly monitored by a sophisticated logging system. Temperatures, flows and levels, both at surface and underground were collected at 15 second intervals and analysed by computer. All important points which were monitored were also protected by an audio visual alarm system and, in the case of the main brine tank levels, should a major brine loss be detected, the underground valves were programmed to close automatically within 15 seconds.

The individual brine flows to each freeze tube were balanced at the commencement of the freeze and then regularly monitored together with the return temperatures to ensure optimum operation.

Creep of the Keuper Marls in the freeze chamber was monitored regularly by means of levelling fixed reference points in the floor and collating same with shaft survey stations. The floor of the Upcast freeze chamber heaved some 10 mm before the commencement of freeze and some 17 mm during the freeze. It can therefore be assumed that the movement that did occur was not freeze induced but rather the natural relaxation of stress in the surrounding formations.

CONSTRUCTION OF SHAFT THROUGH FROZEN GROUND

Excavation

Rates of progress when sinking and lining through the frozen ground averaged between 12 and 15 metres per week depending on ground conditions. This was slower than in the upper sections of shaft because of blasting restrictions and the increased thickness of the concrete lining.

Blasting

It was imperative that blasting operations in the frozen ground did not cause damage to:

a) The shaft brine mains going into the freeze chamber.
b) The freeze chamber installation.
c) The freeze tubes in the ground.
d) The ice wall itself.

Care was taken before sinking recommenced below the freeze chamber to install steel and mesh protection around the shaft brine mains and to build a barrier enclosing the freeze chamber. This eliminated the danger of damage to the installation from flying rock but still left the worry of damage caused by vibration from shock waves.

To minimise this danger the following precautions were taken;

i) Depth of shotholes was limited to 1.5 metres (this was increased to 2.4 metres in certain strata with experience).
ii) The amount of explosive being initiated at any time was minimised by maximising the number of delay detonators used in the blast. This was done by using a mixture of both half second and millisecond delays so that in all 16 different delays were used per blast.
iii) The outer shotholes, (i.e. those nearest to the freeze tubes and the ice wall), were charged with 25 mm diameter explosive sticks as opposed to the 32 mm diameter explosive used in the rest of the holes.

Figure 5 shows the blasting pattern used for a typical 1.5 metre round.

The above precautions proved adequate and no vibration damage was caused. Although some vibration monitoring was carried out in the Upcast Shaft freeze chamber it was not possible to predict at what levels damage to either the installation or the ice wall would occur without testing the system to destruction. This was considered unadvisable!

Backwall Grouting

Backwall grouting of the shaft lining through the frozen section took place in three phases.

Phase one was a low pressure filling operation to ensure that the joints between the lengths of concrete lining were completely filled. To achieve this 6 additional grout pipes were placed in the top of each 6 metre pour angled up into the joint and 12 grout pipes placed at the base of each pour with a ‘T’ piece extension down into each joint. Thus grout could be injected directly into each joint from below whilst air was relieved through the grout pipes above.

The first stage grouting was carried out using a thick S.R. cement mix (200 kg cement to 80 litres water) pumped at a maximum pressure of 10 bar (1 MPa). The exercise was carried out at 2 weekly intervals through the frozen sections so that each joint was grouted whilst the rock/ concrete interface was still unfrozen due to the heat of hydration of the concrete lining.

The second phase of backwall grouting through the frozen zone was carried out some 3 weeks after the freeze had been switched off when temperature monitoring probes indicated that the rock concrete interface had thawed. This exercise was carried out from a gravel filled gravel seal below the frozen section of shaft and progressed upwards to the freeze chamber. Three rings of 12 standard grout pipes had been cast into each 6 metre length of lining and these were progressively drilled out and grouted, working upwards. Grout composition varied according to grout acceptance between 50 to 150 Kg cement per 200 litres grout batched. Injection pressure was limited to 1.25 times the theoretical maximum hydrostatic head.

The third phase backwall grouting took place when a significant increase in the shaft water make indicated that the ice wall surrounding the shafts had eroded away and that the concrete lining was subjected to the full hydrostatic head of the aquifer.

This phase was concentrated mainly on areas of shaft lining that were making water and consisted of a) normal cement injection as per phase 2 followed by b) injection of ultra fine grained cements (600 and 900 grade), and finally c) chemical grout based on Rocagil B.T. grout with additives.

At the time of writing water make in the Upcast Shaft was limited to approximately 0.1 litres per second and in the Downcast to some 0.2 litres per second.

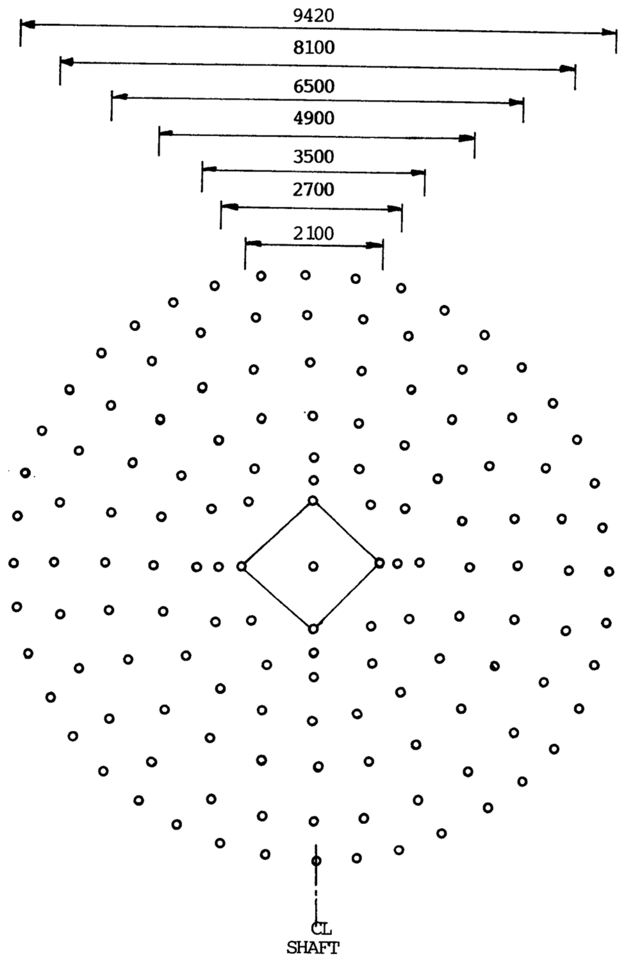
INSET CONSTRUCTION

Both shaft insets were constructed in weak strata. This problem had been recognised by British Coal at an early stage and consideration was given to deepening both shafts in order to re-site the insets in more competent ground.

This consideration had unfortunately to be discounted when information from additional bore-holes confirmed the position of the potentially water bearing igneous intrusion close to the bottom of the Upcast Shaft. Thus the Bunter Sandstone aquifer above and the possible aquifer below gave the planners little choice but to position the Upcast Inset between the two. The Downcast Inset horizon was then constrained by considerations of haulage road gradients to the south. This inset too could therefore not be moved out of the weak Blackshale formation.

Because of the potentially poor ground conditions two fundamental decisions were taken regarding inset construction.

i) To head out from the shaft on a “top cut” level and to construct and inset reinforced concrete roof before benching out the inset to its full depth.



				1.5 m Round		
Ring No	No Hours	Radial Spacing (mm)	Circ. Spacing (mm)	Charge /Ring (Kg)	Carrick Delay No	½ Second Delay No
1	5	-	1480	6.0	0	-
2	8	300	1060	9.6	2	-
3	12	400	900	14.4	4,6	-
4	20	700	800	24.0	8,10	-
5	24	800	850	28.8	-	1,2,3
6	28	800	900	33.6	-	4,5,6
7	40	660	740	31.2	-	7,8,9,10
	137			147.6		

Note:
1. Rings 1 to 6 on 32 mm dia. cartridges
2. Ring 7 on 25 mm dia. cartridges
3. All Rings on S.G. 80
4. Control hole charged as per cut
5. Minimum Intershot delay 60 millisecs.

FIG. 5 SHOTFIRING PATTERN

ii) To minimise the effect on the strata of blasting by the introduction of roadheader type heading machines into the insets as soon as practically possible.

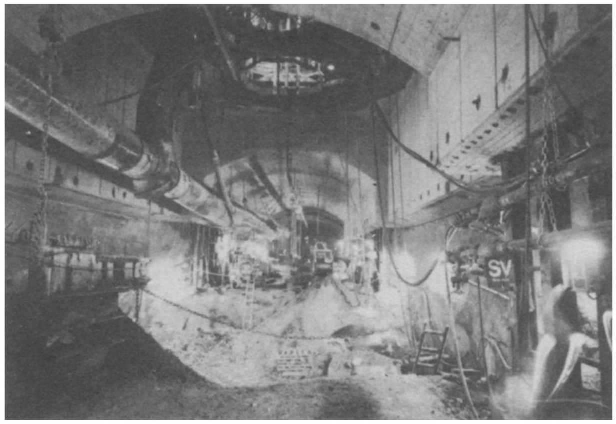


Fig.6

The construction of the insets was therefore done in the following sequence;

Excavate inset crib in shaft approx. 6 metres above the top of the inset.
Install 40 mm threaded bars from crib to roof of inset box section top.
Sink into top of inset box.
Open out top of inset box locating 40 mm threaded bars in roof and using for temporary support.
Create top of inset box.
Excavate box down into middle of inset.
Break out on top cut level and drive out 25 metres each side of shaft centreline.
Construct reinforced concrete inset roof.
Bench down to inset floor level, install roadheaders and drive out into inset roadways (see Figure 6).
Concrete floor and walls of inset roadways. Sink below inset box section into top of sump.
Concrete top of sump and remainder of inset box section.

Excluding the time spent on periods of back-wall grouting in the shaft during inset construction and the initial pit bottom roadway drivage, both insets were completed in approximately 16 weeks each.

CONCLUSION

At time of writing, the shaft sumps below the insets had not been completed and will not be completed until the initial underground roadway drivage is finished. Both shafts will then be furnished with the permanent winding equipment and minerals and materials handling facilities.

However, on taking into account extensions to the programme for additional work, e.g. backwall grouting, at completion of the insets, the Upcast Shaft was some eight weeks ahead of programme and the Downcast Shaft twelve weeks ahead.

This achievement reflected the reliability and dependability of the freezing process as a method of ground treatment and the determination of both British Coal and the Contractor whilst breaking no records to maintain a steady and planned rate of progress throughout the many and varied phases of the sinking of the Asfordby Shafts.

ACKNOWLEDGEMENTS

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Precast concrete segmental lined shafts

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SYNOPSIS

Tunnels for railways, roads and everyday services use shafts as an aid to construction and are not necessarily a permanent feature of the finished scheme. As a result their development and means of construction has lagged behind that of tunnels.

The Paper describes various construction methods and their evolution brought about by economic needs and the necessity to ensure an acceptable end structure to meet the clients requirements.

Market pressures required lining manufacturers to examine their products, improve designs and introduce new systems to assist the client in reducing his costs.

The aim must be for this development to continue into the future as a major element of the tunnel industry.

PRECAST CONCRETE SEGMENTAL LINED SHAFTS

All new cost-effective systems used in the construction industry take time to evolve and are not immediately accepted. Reinforced concrete segmental shaft and tunnel linings were the exception but so were the circumstances which caused the change.

Reinforced concrete bolted segmental tunnel linings were developed as an alternate to cast iron tunnel linings resulting from re-armament prior to the last world war creating a shortage of this material and were first used to construct the Ilford Extension in 1937–38. Cast iron linings had been in use for many decades before this change therefore the principle of segmental linings to build shafts and tunnels was not a new concept. It was unfortunate that concrete was chosen as the replacement material because it was not ideal for a structure dependent upon flanges and bolts for its ultimate stability. Having made this statement, the design evolved over the years and many miles of tunnel have been constructed using the original principle.

As concrete segmental linings became more widely used in tunnel construction, cast iron and timber-lined shafts were slowly replaced by purpose-designed precast bolted concrete segmental linings. Shaft development did, however, lag behind for many years and it has only recently started to catch up with new improved designs and construction techniques.

The first concrete segmental shafts were used to build a sewer tunnel constructed by Kinnear Moodie Company Limited in 1946–47. The shafts were 12'0" I D, 13'0" 0 D and 2'0' wide with six segments plus key to complete the circle. The segments were bolted together through flanges and the shafts sunk as caissons.

A 6'5" I D precast concrete segmentally-lined tunnel was driven from the shafts through water-bearing ballast with a standard hand shield. The primary segmental rings were finally lined with engineering bricks to produce the finished sewer.

DESIGN

Linings for shafts are designed on the assumption that the load is evenly distributed around the structure and increases with depth.

In the early stages of development, when bolted flanged linings were used, the Engineer had to design on the basis of a lining capable of supporting the maximum load at the bottom of the shaft.

Since early shafts were relatively shallow, this was not financially significant but as the system became more widely used and shafts became deeper, standard designs were produced for depths of about 30 metres.

To use the linings at greater depths, the cross section area must be increased and this is achieved by extending the external diameter of the lining thereby increasing the skin thickness and the cross section area. The solution is only cost and structurally effective up to an increase of 150mm which enables the lining to be used to a further depth of approximately 15 metres. Beyond 45 metres it becomes necessary to re-design the lining completely.

In the late 70's the position changed with the introduction of the Smoothbore system which gave the Design Engineer a uniform section in which to vary structural requirements. This development will be discussed later in the Paper.

The equal loading principle changes when an opening has to be formed for tunnel exit or entrance and, depending upon the diameter of shaft and tunnel, ground conditions and depth at which the opening or openings are required, determines the hoop stresses from the partial rings that have to be transferred around the opening if local failure is to be prevented. Methods of achieving this requirements are discussed later in the paper.

METHODS OF CONSTRUCTION

Segmentally-lined shafts are constructed employing either or both of the following methods, underpinning or caisson. Each method has a number of variants and refinements and the main systems are described below.

UNDERPINNING:

The method is mainly employed where shafts are sunk through good cohesive soils with little or no ground water.

The operation starts with a circular excavation to a depth equal to the width of two or three rings of lining. Timber foot blocks are placed around the bottom periphery of the excavation and levelled in. From this level base, the segments are built chimney fashion up to the surface. It is important that the rings are erected true to shape and level as subsequent rings will be affected by any deformation once the underpinning commences. See Figure 1.

Depending upon the size of the void behind the initial rings, it will be filled either with cement grout or concrete. Before excavation recommences, the grout or concrete must be capable of anchoring the lining to the ground thereby supporting the weight of subsequent sections of lining as the shaft is sunk.

The method of excavation is dependent upon the diameter of the shaft. Generally speaking, diameters between 2.44 m and 5.00 m are excavated and loaded by hand into skips but with larger diameters mechanical aids are employed.

The erection of the lining is achieved as a combined operation between the miners and surface crane. The segments are lowered to the shaft bottom and manouvred into place by the miners for connection onto the previous ring of lining. To achieve this operation special hooks are employed which are detailed in Figure 2.

The rings must be grouted at least once a shift, or more often if the ground is suspect. In any event not more than three rings should be left hanging as the dead load may cause circumferential flange failure at the last grouted ring.

The process described above continues until the number of rings built achieve the prescribed depth. To complete the shaft, an insitu concrete base is cast into the shaft bottom which usually includes the last ring of lining in its depth.

UNDERPINNING USING COMPRESSED AIR:

This expedient is used in water-bearing ground such as silts, sand and gravels where the air pressure holds the water in check stabilising the ground and enables excavation to be carried out normally. The method is expensive and has a considerable element of risk if the air decks, locks and kentledge are not adequately designed and correctly installed. Considerable experience and expertise is necessary to carry out this method. It is, with very few exceptions, being replaced by controlled caisson construction methods described later.

SHAFT SHIELD:

The occasion may arise where a limited depth of non-cohesive ground is encountered sandwiched between cohesive soil. Provided this situation is known, a shaft shield can be installed whilst excavation is taking place in the stable ground and used to traverse the water-bearing section.

The shield, a simple steel cylinder as shown in Figure 3, is equipped with a cutting edge and circumferential rib to provide rigidity plus support and reaction surface for the manually-operated jacks located around the periphery. The shield is forced downwards during excavation by the jacks reacting against the last ring of linings built. The segmental lining is erected within the protection of the tail employing standard shaft lining erection techniques as already described. The process continues through the poor ground into the cohesive soil where the shield is abandoned by removing the rib, building linings through the steel cylinder and finally grouting the void between the two.

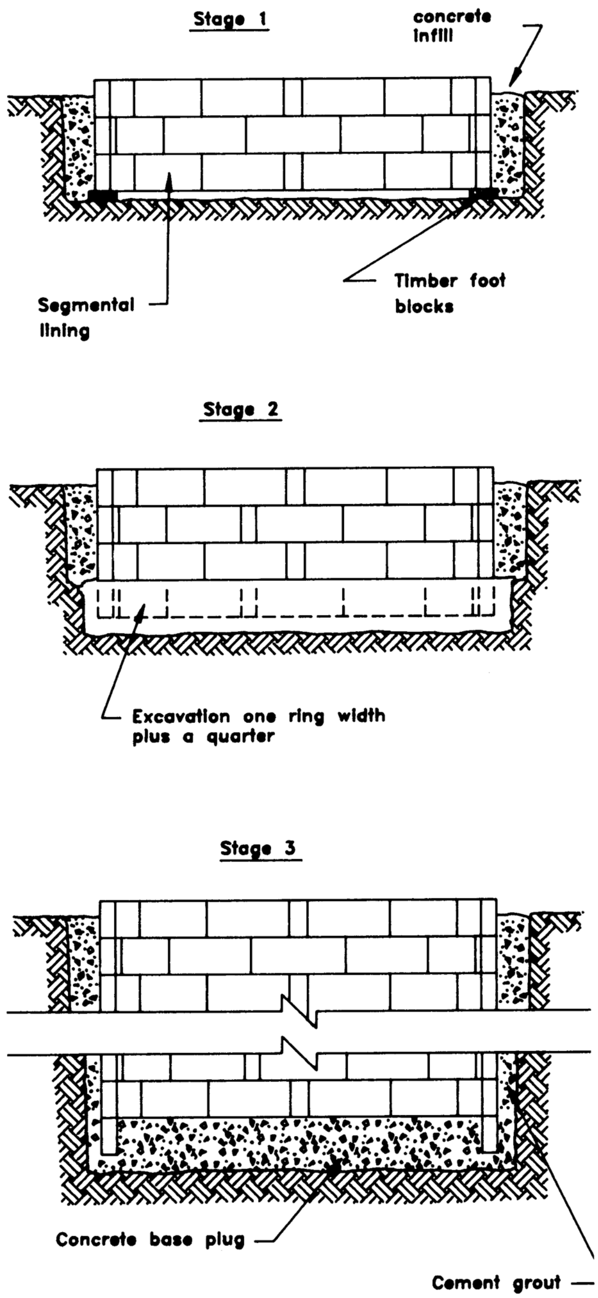
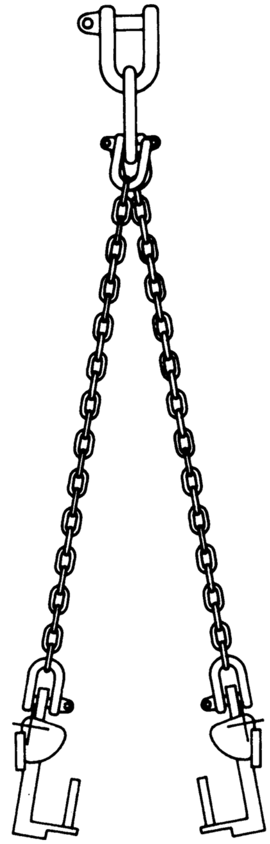


Figure 1 **Shaft sinking using underpining system.**

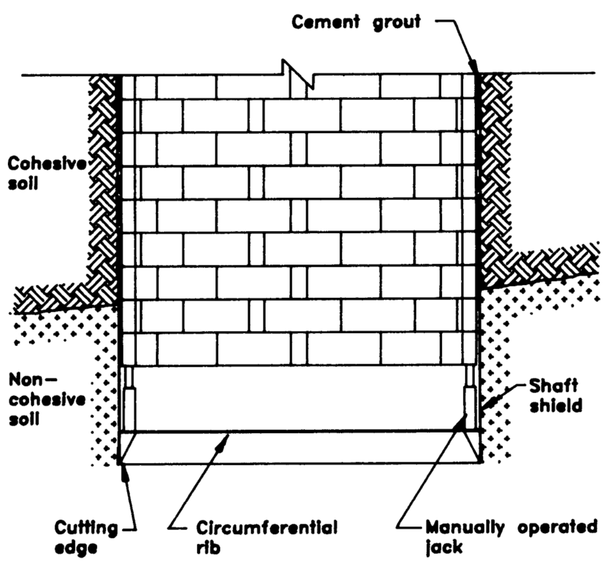
CAISSON SINKING

The construction of shafts using the caisson method has always been associated with unstable ground and from the original crude but effective rough-and-ready system has developed, over the last twenty years, into today's controlled sophisticated construction process.

The method originally employed was to erect a cutting edge and choker ring in open excavation and then build additional rings chimney fashion above ground level to a pre-determined height and erect a steel kentledge support frame to the top of the last ring. It is usual to stagger the longitudinal joints to stiffen the structure except where openings have to be formed when employing this method. When a bolted lining is being used, care must be taken to ensure that the support frame is located over the longi tudinal stiffeners in the segments. Kentledge, distributed around the circumference of the shaft, is loaded onto the frame leaving an opening large enough for the grab to pass through. It is important that the circularity of the cutting edge and choker ring are maintained throughout the sinking process because any deflection will increase as the shaft is forced downwards into the ground causing progressive upward deformation resulting in eventual collapse of the lining. For this reason the design of the cutting edge must take into account the lateral forces from the ground during the sinking process.



<u>Figure 2</u> Segment erecting chain hooks

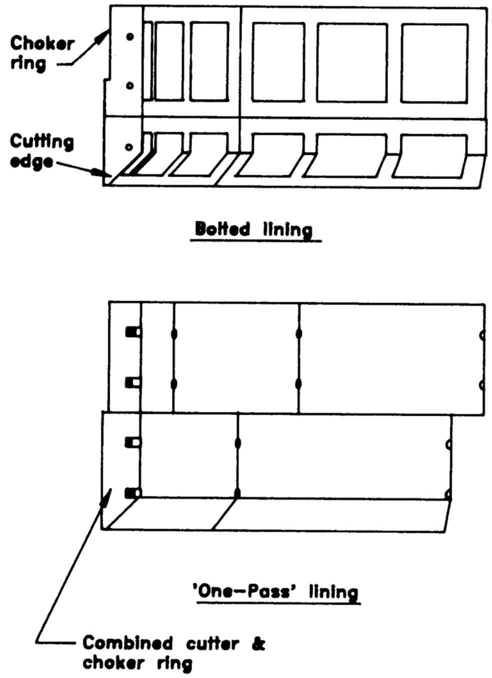


<u>Figure 3</u> Shaft sinking using a shaft shield.

Excavation is normally carried out by a crane-operated grab and the shaft slowly sinks into the void created due to its own weight and that of the kentledge, the cutting edge trimming the sides as downward movement proceeds.

When the top ring of lining reaches ground level, the kentilage and frame are removed and the shaft lining extended and subsequently re-loaded to continue the construction process. The sequence continues until the shaft has reached its design depth or has traversed the unstable ground and bottomed into cohesive strata.

The level and verticality of the shaft is maintained either by the pattern of the excavation or by the distribution of the kentledge around the circumference of the shaft or both.



<u>Figure 4</u> Cutting edge & cheker rings

During the sinking operation, the cutting edge must be kept buried at all times as failure to achieve this may cause a run of ground creating a void behind the lining resulting in unequal loading and subsequent distortion of the rings and the possibility of the shaft moving out-of-plumb.

The main problems with this method is the friction between the lining and the ground which restricts the depth to which the shaft may be sunk and the limited control possible over verticality and level.

It is normal when sinking through unstable ground for the excavation to be carried out using a grab and any ground water allowed to flood the shaft as the sinking operation proceeds. The head of water assists in stabilising the bottom during construction preventing major earth runs and helps-balance eccentric loadings on the lining during construction.

If the shaft is founded on non-cohesive water-bearing soil, it is necessary to maintain the head load of the water in the shaft to prevent the bottom blowing. The concrete plug has then to be placed through the water using a trémie and finished off when the shaft has been pumped dry.

Another factor which must be considered under these conditions is the phenomena of flotation which exerts an upthrust on the structure once the plug is in place and any water pumped out. The design must ensure that the dead weight of the shaft lining, plus the plug, is greater than the upthrust of the ground water. Any subsidiary elements such as landings, cover slabs or superstructure must not be considered in this calculation.

Shaft linings are made up of Ordinaries, Tops and Key Segments. The circumferential dimensions are so arranged that two ‘ordinary’ segments are equal in length to two top segments plus a key. This feature is a requirement for underpinning and opening forming but, when the shaft is being sunk as a caisson, where erection of the lining takes place in the open, the key and top segments may be replaced by two ordinaries.

In water-bearing ground this reduces the number of joints to be made water-tight and also produces a stiffer structure.

The situation does arise where the shaft has to be sunk through unstable ground overlying cohesive rocks. Both soils require different construction techniques, caisson and underpinning. The difficulty arises at the interface between the soil types. Before underpinning can take place, the cutting edge and choker ring must be completely buried into the cohesive soil and the section of constructed shaft efficiently grouted. The cutting edge may now be removed in comparative safety and this operation is assisted by the design of the choker which, apart from creating a larger void during caisson sinking to reduce skin friction, prevents the possibility of a run of non-cohesive ground during this operation. See Figure 4.

The caisson method of shaft sinking was greatly assisted in the early 70’s by the use of Bentonite slurry to form a lubricating skin between the shaft lining and the ground. The general principle is shown in Figure 5.

As shown an insitu concrete ring wall, 100–150mm greater than the outside diameter of the shaft lining, is built to support the ground for a depth of approximately 1.5 metres. The cutter and choker rings plus the additional rings and kentledge as already described are erected and the void between the insitu concrete and shaft lining filled with Bentonite slurry and maintained full as the sinking operation proceeds.

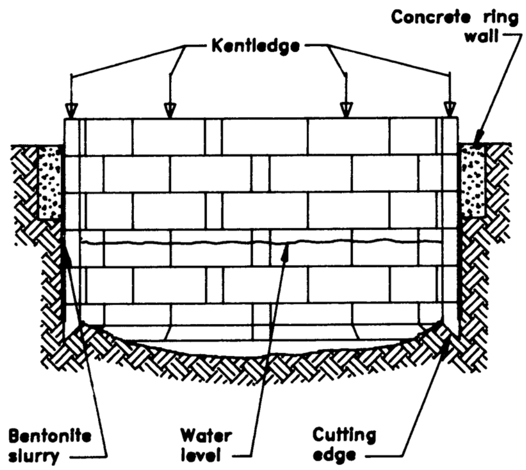


Figure 5

The slurry has three functions, it reduces the skin friction between shaft lining and the ground, it maintains the void created by the choker ring and also acts as a barrier reducing the inflow of water into the shaft through the segment joints.

This method enables shafts to be sunk to greater depths than previously possible and in many cases eliminated the use of compressed air where previously this had been the only solution.

Since this development, the method has been refined to give greater control to the operation in maintaining verticality, shape and final level. These improvements are shown in Figure 6.

It is important to remember that for structural stability any voids behind the lining must be pressure-grouted once sinking is complete before proceeding with the next phase of construction.

CONSTRUCTION OF OPENINGS

It is prudent when using segmental linings in shaft construction to stagger the longitudinal joints thereby assisting maintenance of circularity and stiffening the overall structure. The exception is at openings. To form an opening, segments have to be removed from the shaft lining either by cutting out mechanically or by removing a given number of segments adjacent to keys. To achieve this latter course, which is recommended, shaft linings have been designed so that the circumferential length of two top segments, plus a key, are equal in length to two ordinary segments. Therefore when the level for the top of the opening is reached, the staggering of the joints should change to building in line over the depth of the

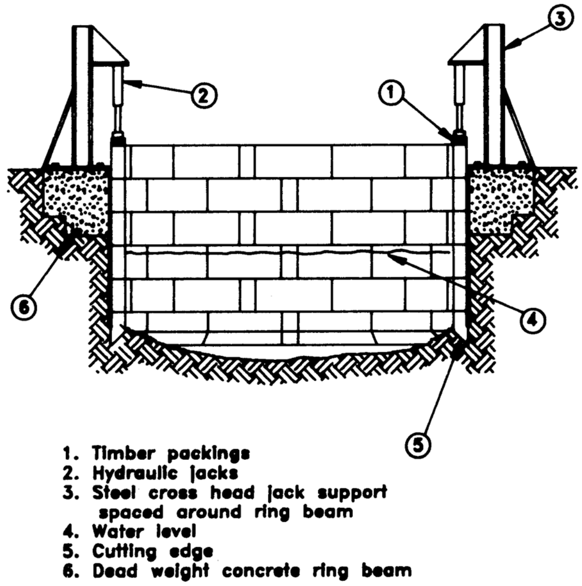


Figure 6 Caisson sinking using jacks to replace kentledge

opening. If the shaft has been orientated correctly, the key will lie on the centreline of the tunnel. To form the opening the keys and a number of segments depending upon the size required are removed. It is assumed that the shaft has been supported over this section to prevent any movement during this operation. A typical support method is detailed in Figure 7.

Where solid linings are employed such as the 'One-Pass', the change from staggering of joints is not necessary as segments for use at openings are supplied as standard. When more than one opening is required at the same level, two top plates plus a key may be substituted for two ordinary segments at the appropriate position in the ring. To form openings of this type, the diameter of the shaft must ideally be twice the external diameter of the tunnel lining. See Figure 8.

When forming an opening in a shaft, the existing and anticipated hoop stress from the partial rings must be transferred into the structure above and below them. In small diameter shafts in stable ground, this can be achieved with mass concrete between the shaft and tunnel but as the diameter of the shaft and tunnel increase, more sophisticated structures are necessary. In these cases steel jambs are bolted onto the longitudinal joint faces of the segments adjacent to the opening to transfer hoop loads to the mass concrete structure which is poured around the opening and the tunnel linings. A typical opening is shown in Figure 9.

The design of the opening is dependent upon the diameter of shaft, exit or entrance tunnel, ground conditions at opening level or the number of openings to be formed. In the author's opinion, very little thought is given to what can be a critical part of tunnel construction.

IMPROVEMENT IN DESIGN

It is a well known fact that concrete is an ideal economic material in compression but requires expensive steel reinforcement to cater for any tension.

With a structure like a shaft where the ground loading is uniform and only varies with depth, it makes sense to design a lining system which takes advantage of the characteristics of the concrete rather than the steel.

The development of methods to connect segments together without the need of bolts and flanges has produced linings of uniform cross section which in turn has given the Engineer more flexibility to produce economic designs which cater for changing loadings as the depth of the shaft increases.

Comparing the traditional bolted lining with one having a uniform cross section, the following advantages are apparent:

The thickness of the lining is reduced by 3% for 5.00 m diameter increasing to 28% for 9.0 m diameter linings.

Because the reinforcement can be distributed more effectively, the steel content is reduced resulting in savings in supply and its fabrication.

The changes discussed above, coupled with improved construction techniques, have resulted in rethinking the number of segments and their width which make up the unit depth of shaft lining. The trend is towards a smaller number of segments of greater width, creating in turn a more economic structure.

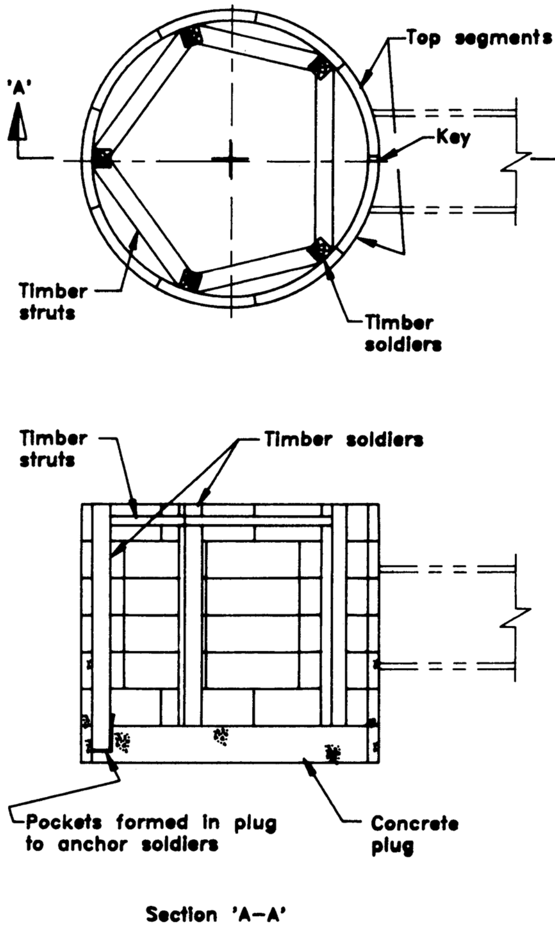


Figure 7 Shaft support for opening construction

CORBELS

When shafts are used for future access to the tunnel, ladders and landings have to be installed. This has been achieved either in flanged, bolted linings by support rings built into the circumferental joint during construction or formed in the secondary lining when one is required.

The latest solution is purpose-made corbel segments which are built into the shaft at the appropriate level as construction proceeds. This system is unique to the 'One-Pass' Lining. See Figure 10.

SOFT EYE OPENINGS

As already described, the formation of an opening in a shaft in unstable ground conditions is expensive and not always completely successful.

A recent development has been a precast concrete shaft eye which is designed for use in either underpinning or caisson shaft sinking.

As will be seen from Figure 11, the Eye unit is in two sections. The main structural element is manufactured from standard reinforced concrete but the concrete forming the temporary filling of the eye is low strength low impact concrete. Once the shaft has been completed with the plug in place, no further work is required at the opening other than to remove the soft concrete.

The introduction of this new precast addition to the segmental shaft lining will eliminate the temporary works at the shaft bottom and subsequent insitu construction resulting in lower costs, reduction in contract time to the benefit of both client and contractor and a better engineered structure.

When the tunnel has to exit into water-bearing ground, a circular gasket is bolted onto the face of the opening. The tunnelling machine, which in this instance is assumed to be slurry or earth-balancing type, enters through the gasket and commences to excavate through the soft concrete. The gasket maintains the slurry around the machine during this operation allowing it to enter the ground and commence building the tunnel.

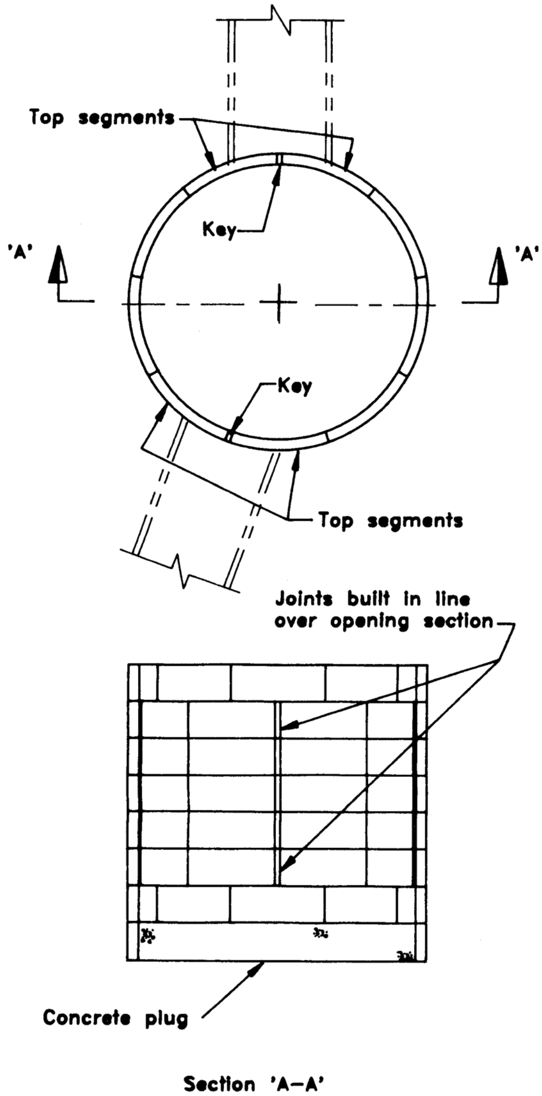


Figure 8 **Forming two openings at same level**

WATERPROOFING SHAFTS

Waterproofing precast concrete shaft segments has always been a problem when water-bearing ground has been encountered. The only practical solution before the introduction of preformed gaskets was an asbestos cement impregnated cord called 'Philplug' caulked into the joints.

The material had one major drawback, its lack of flexibility once the cement had set. Any movement in the structure would spring the joint causing secondary leaking requiring cutting out and recaulking.

The material, however, successfully caulked many miles of tunnel and shafts in the U K and overseas before it was ultimately withdrawn from use because of its asbestos content.

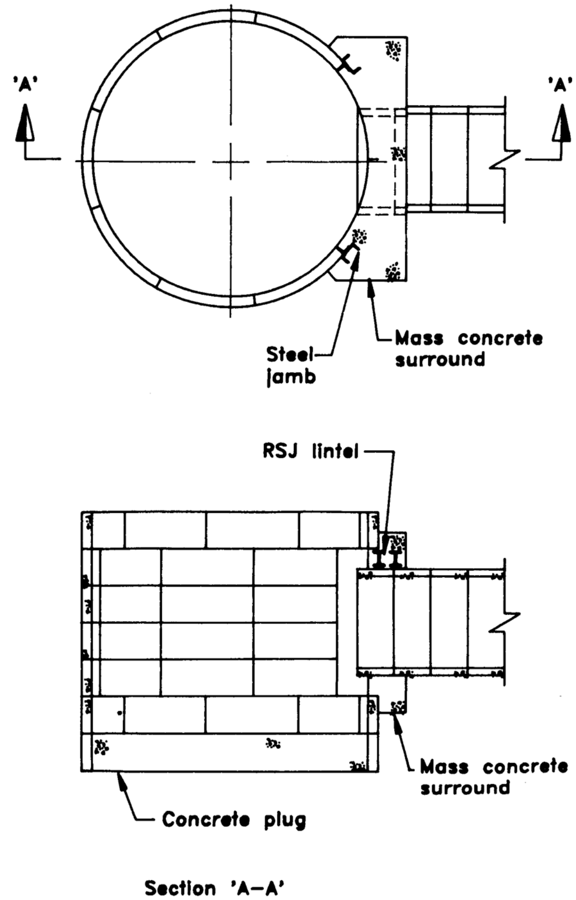
GASKETS

The universal acceptance of reinforced concrete shaft and tunnel linings created a demand for stringent specification of water-tightness. This requirement further increased with the introduction of the smoothbore one shot lining.

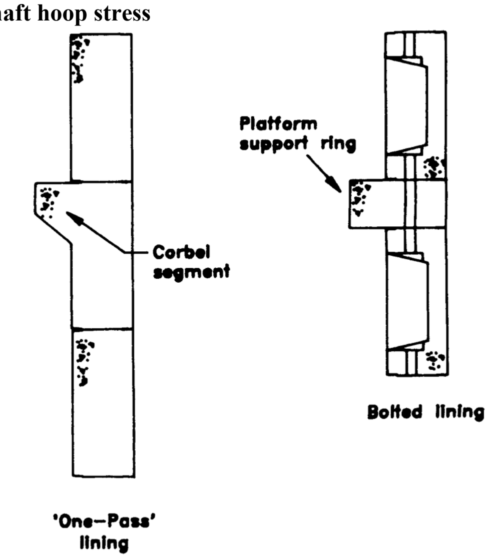
To meet the new specification, preformed Neoprene Gaskets developed in Germany but were not introduced into the U K until the withdrawal of Philplug.

The gasket is formed from an extruded Ethy lene-Propylene Terpolymer (EDPM) or Stirine-Butadiene Rubber (SBR), the section of which is generally as shown in Figure 12.

The finished gasket is rectangular in shape produced from a combination of the extruded section and purpose-made corners, the whole being vulcanised together. The gaskets are designed to fit into a groove cast around the periphery of the



<u>Figure 9</u> **Standard structure to support shaft hoop stress**



<u>Figure 10</u> **Platform support systems**

segment the section of which is critical to the efficiency of the gasket. The groove must have a cross section area equal to that of the gasket so that the material will compress completely into it when total closure is achieved.

These gaskets seal as a result of rebound pressure exerted on adjacent faces achieved by the interaction of the connecting mechanism between segments.

The main problem with this system is great care must be taken during the erection of the lining to ensure watertightness. If leaking does take place the only remedy is secondary caulking.

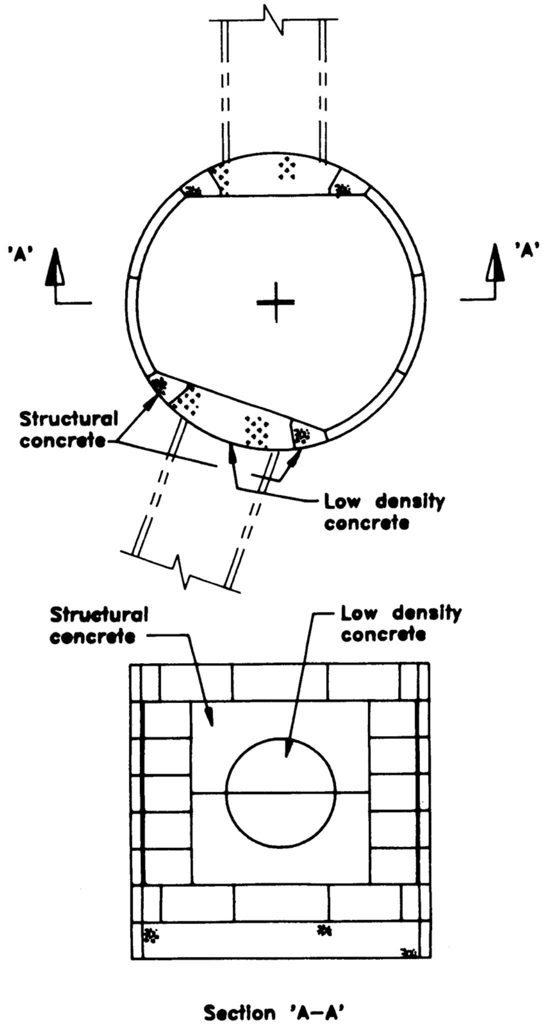


Figure 11 **Soft eye openings**

HYDROPHILLIC NEOPRENE

Running parallel with this development was research into Hydrophillic Neoprene. As a result there are now two types of gasket available on the market.

Hydrotite is a modified neoprene which expands ten times its volume when it contacts with water. The gaskets are produced from extruded strip generally as Figure 12. As a result of the swelling properties of the material, the cross section of the extrusion is much smaller which in turn eliminates the need for special mitred corners, the resulting assembly and vulcanising problems. The gaskets are assembled on site by unskilled labour from material delivered in 20 metre lengths. The complete gasket is housed in a groove but due to the swelling property of the Hydrotite its design is not as critical as for the Ethylene-Propylene Terpolymer (EDPM) or Stirine Butadiene Rubber (SBR). The material seals in the first instance as a result of pressure from the segment connecting mechanism. If a leak does materialise the swelling properties promptly choke off the flow of water. The material is more construction tolerant and does not require excessive skill to produce an effective seal.

USES FOR SHAFTS

Shafts are used in the main to provide access to underground workings but in recent years additional uses have been found:

Pump chambers.
Wet and dry wells.
Piles for tall buildings.
Constant temperature chambers for scien

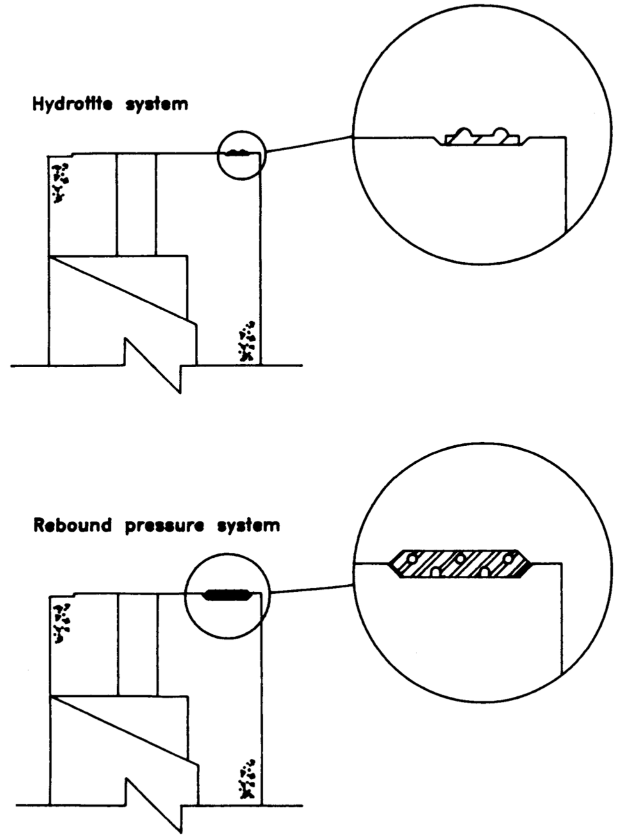


Figure 12 Preformed gaskets

tific instruments.
Storage tanks
Filter beds.

NEW SHAPES

Segmental shaft linings have been designed to produce circular structures. This shape is not always the most convenient for some tunnelling systems, i e pipe jacking. Ideally a long narrow shaft is more space-effective to house the jacks and pipes. Such a solution is being developed and will shortly be available on the U K market.

CONCLUSIONS

Over the last fifty years reinforced concrete segmental linings have developed from a tunnel lining built vertically to today's purpose-designed structures.

Suppliers now have systems which enable the Engineer and Contractor to purchase precast elements which eliminate all insitu concrete with the exception of the base plug.

Due to improvements in design, manufacture of linings and construction techniques, shafts are being built to greater depths with larger diameters through water-bearing noncohesive soils with greater safety and speed.

It is unfortunate, however, that the techniques described in the paper are not in universal use and there may be a case for specialist contractors to be considered when difficult shafts have to be constructed.

Geotechnical risk assessment for large-diameter raise-bored shafts

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SYNOPSIS

A significant increase in the diameter of raise bored shafts has been seen recently with advances in techniques and equipment. With the increase in diameter there is a greater potential for instability of the shaft walls and the advancing face of the raise.

Although raiseboring minimises the disturbance of the rockmass, the ground conditions must be adequate to provide inherent stability for the raise diameter proposed. Raiseboring of shafts in excess of 4m diameter requires at least 'fair' quality ground conditions or better as determined by rockmass classification methods.

The paper presents a probability chart from which the compatibility of raisebore rock quality QR and proposed shaft diameters can be assessed. The chart is based on an adapted form of the Q-system of rockmass classification. The method of analysing the reliability of a raise-bored excavation from the distributions of the rockmass parameters is documented. The acceptability of a particular probability of failure is discussed.

A flow chart of geotechnical evaluation activities required to determine the significant parameters for assessing raise stability is presented. The sufficiency of geotechnical information for raisebored shafts is discussed in terms of the variability of the critical rock-mass parameters and the specifications of a proposed shaft.

In recent years the state of the art of rock boring has developed significantly with improve ments in techniques and performance of boring machines.

Raiseboring has paralleled these advances in techniques and size. In the early 1980's, raisebored holes in mining generally ranged between 1,2 m and 2,44 m (4 ft and 8 ft), with the maximum diameter being 3,6 m (12 ft). Technological advances since then have been such that raisebored shafts in excess of 6,0m are now possible.

With this increase in diameter of raisebored holes however, comes greater potential for instability of the walls and advancing face of the raise. Whereas the 1,2 m and 2,44 m raisebore shafts would be self—supporting even in poor ground, 6 m raisebore shafts create a significant perimeter length (19 m) and a large undercut area (28 m^2). Although it is true that boring generally minimises the disturbance of the surrounding rock mass as compared with drilling and blasting, the ground conditions through which the shaft will pass require to be of sufficient quality to allow a free standing, unsupported excavation. It is therefore an essential prerequisite that the in-situ rock mass conditions are known prior to excavating the raise. This refers mainly to the structural and strength features of the rock mass. The in-situ stress conditions should also be investigated together with the likely changes in stress conditions induced by mining which also require to be accommodated.

This paper discusses the potential risk associated with raisebored shafts with respect to the size of the shaft and the method of boring. It presents guidelines for investigation activities required including the available sources of information to define the ground conditions. A chart has been prepared which allows a quantitative assessment of the risk attached to any shaft prior to commencement of the excavation so that the reliability of the proposed system can be evaluated.

COMPARISON OF RAISEBORING TO CONVENTIONAL SHAFT SINKING—EFFECT ON ROCK MASS AND GEOTECHNICAL RISK

Regarding the effect on the immediately surrounding rock mass, the following advantages and disadvantages are recognised in comparing raiseboring over conventional shaft sinking by drilling and blasting.

Advantages of raiseboring

- Damage to rock mass behind excavation is minimised
  - maintaining an inherent rock strength and –stability.

- Smooth finish on shaft obtained
  - precluding the requirement for lining and an important consideration in ventilation shafts.

Disadvantage

- **Remote excavation face**

– no access for remedial support installation.

A raiseboring operation can be described as 'remote' as the methods excludes access to the working face. This is in contrast to conventional shaft sinking which is a 'hands-on' operation. The major disadvantage of the remoteness from the excavated face is that it precludes immediate ground support on intersection of poor ground conditions. In conventionally sunk shafts, rock support is included as part of the excavation cycle with early access to the sidewalls from the floor of the excavation.

The vertical nature of raisebored shafts also makes them different from bored tunnels as access for support purposes is usually available in the latter behind the advancing face.

The implication of the remoteness of the raiseboring operation is that the conditions of the rock mass in which a shaft is to be excavated should be defined to a sufficient degree of accuracy so that the compatibility of raiseboring of a certain diameter and rock condition can be evaluated. Unexpectedly adverse ground conditions cannot be dealt with successfully by remedial measures available in conventionally excavated shafts may force the abandonment of a raise.

A geotechnical investigation for conventional shaft sinking, is usually concerned with location, support requirements and groundwater inflows. Cover drilling provides on-going geotechnical information during sinking. For raiseboring however the thrust of investigations are for overall feasibility of the shaft. The risk to overall feasibility in conventional sinking is much lower than with raise boring. It is important that the risk to raiseboring is accurately defined with appropriate pre-excavation investigations.

RAISEBORING METHODS

Two different raiseboring techniques are currently utilised for large diameter shafts:

- Conventional i.e. full face raiseboring
- Sequential raiseboring

In full face raiseboring a single cutter head is raised from the bottom to the top station. The cutter discs or inserts describe circles in the face with the disc configuration such that the rock fails mainly in shear, forming small chips of rock. The raisebore head may be one of several types dependent on the configuration of the cutters but the essential aspect is that the head is confined through approximately 270 degrees.

The sequential raiseboring technique utilises two cutting heads, an upper head similar to a conventional single head and a lower 'reaming' head. These cut alternately, or 'sequentially', on short cuts (less than 1 metre) with the other head disengaged from the face. The relative diameters of each of the heads is designed so that the cutting area of both is equal. Similar torque and thrust is therefore required to drive and bore for each head.

Upper head operates with similar rock confinement to the conventional head. With the lower 'reaming' head however the cutters are largely acting on a brow' of material with 270 degrees of freedom and only 90 degrees of rock confinement. The action of the cutters has a 'point loading effect. On the upper head this only produces local shearing of the rock to produce chips. In the lower head however, this point loading could produce shearing through intact rock causing failure of the brow. In poorer ground conditions, ie lower strength and/or closely jointed rock, this situation is exacerbated with significant failures possible from the brow area.

The actions and conditions of full face and sequential head raiseborers are shown in figures 1a and b.

SUITABILITY OF GROUND CONDITIONS TO RAISEBORING: GEOTECHNICAL FACTORS

Whereas raiseborer specifications, ie method and machine characteristics, operator experience and raise geometry (diameter and length), have significant bearing on the success or otherwise of an excavation, the most important factors related to risk are geotechnical in nature.

The main geotechnical factors are:

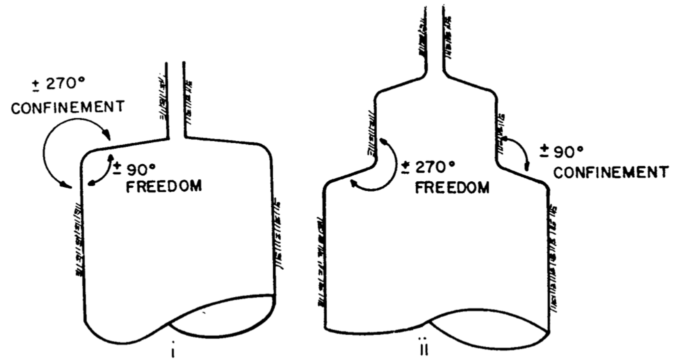

FIG 1a RAISE CONFIGURATIONS i. CONVENTIONAL 11. SEQUENTIAL HEAD

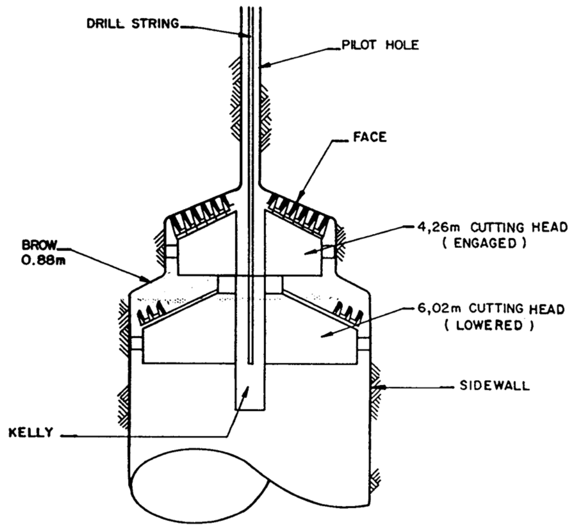

FIG. lb SEQUENTIAL HEAD RAISEBORE CUTTING ACTION

- lithology
- major geological features: dykes, faults, folds, contacts etc.
- rock fabric: discontinuity, orientation, spacing, persistence.
- Discontinuity shear strength: joint roughness, infill, alteration, water.
- Rock strength: intact strength, weather-ability.
- Groundwater.
- In-situ stress.
- Change in stress

The common modes of failure in raiseboring are :

– face failure sidewall failure
– sidewall failure

Figure 2 illustrates these conditions. These may be caused by gravity falls or slides of joint defaced blocks or wedges or by plucking by the cutters. Dependent on the size of the failure, both can cause severe damage to the cutter head and affect stability and effectiveness of the shaft.

ROCK MASS CLASSIFICATION FOR RAISEBORED SHAFTS : Q_R RAISEBORE QUALITY, (a modified Q System)

A number of rock mass classification systems are available to combine geotechnical parameters and quantify rockmass quality and excavation stability.

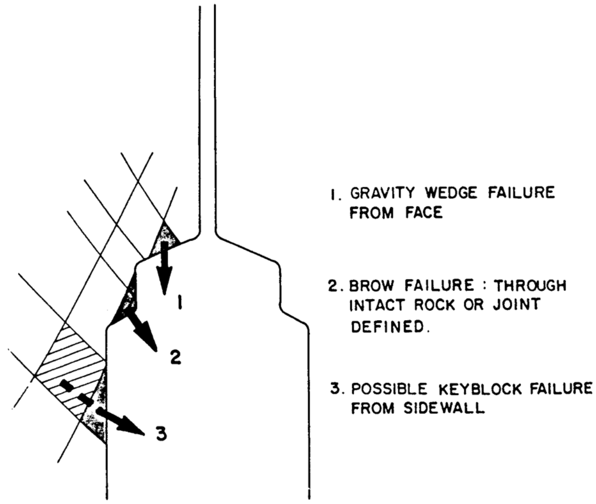

FIG 2. POSSIBLE MODES OF WALL AND FACE FAILURE

The Norwegian Geotechnical Institute 's Tunnelling Quality Index (Barton's 'Q' System[1]) is considered well suited to assess the classification of the rock mass quality for raiseboring. The Q System with Kirstens[2] approach to determine the SRF value is proposed as the basis on which to determine Raisebore Quality, QR.

The Tunnelling Quality Index Q is obtained from:

$$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF}$$

Where:

It should be noted that the Q value is itself simply a rock quality indicator. Additional conditions having a significant bearing on raisebore stability are:

- The shaft wall as opposed to the excavation roof controls final stability.
- The orientation of the shaft with respect to structural features.
- The weatherability of the rock.

To obtain the Raisebore Quality index, Q_R, the following adjustments for Q for the above conditions have been found to be acceptable in practice.

• WALL ADJUSTMENT

The Q System is mainly concerned with tunnel roof stability. In raiseboring it is the wall stability which is of more importance to the final excavation. The following adjustments should be made:

Q sidewall	2.5Q	where	Q>1
Q sidewall	Q	where	Q≤l

Figure 3 shows the normal form of the Q System chart:

RQD	=	Rock Quality designation (Deere 1963)
Jn	=	Joint Set Number
Jr	=	Joint roughness number
Ja	=	Joint alteration number
Jw	=	J water reduction factor
SRF	=	Stress Reduction Factor
$\frac{RQD}{Jn}$		gives an estimate of rock block size
$\frac{Jr}{Ja}$		gives an indication of discontinuity shear strength
$\frac{Jw}{SRF}$		indicates the conditions of active stress surrounding the excavation.

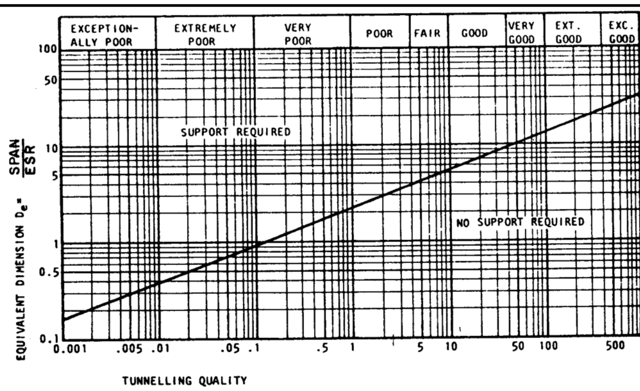

FIG. 3 Q SYSTEM - ROCKMASS CLASSIFICATION

• ORIENTATION ADJUSTMENT

Of major Importance is the orientation of the raisebore axis with respect to dominant or persistent discontinuities eg. bedding, schistocity. The orientation of major discontinuity sets has a different effect on the raise sidewalls as compared with the raise face.

– Face Orientation Adjustment

No of flat dipping (0°–30°)	1	2	3
Major joint sets			
Adjustment of Q	0,85Q	0,75Q	0,60Q

– Wall Orientation Adjustment

No of steep dipping (60°–90°)	1	2	3
Major joint sets			
Adjustment of Q	0,85Q	0,75Q	0,60Q
(The above assumes that the raise shaft is vertical)			

These are simply convenient adjustments and do not replace detailed stability analyses which should be carried out to assess the stability of potential failure blocks and wedges which are likely to occur.

• WEATHERING ADJUSTMENT

The weathering potential of the rock mass should be considered where this will effect the raise stability in the longer term.

Adjustments for weathering are partly accommodated in the Ja (joint alteration) number of the Q system. The following further adjustment for long term sidewall weathering which may affect the intact rock is suggested.

Weathering Index	slight	moderate	severe
Adjustment of Q.	0,9Q	0,75Q	0,5Q

CUMULATIVE ADJUSTMENTS

The above adjustments are cumulative. The deterioration of the raisebore quality QR using the adjustments is given by way of the following example. Sidewall, 2 steeply dipping joint sets, moderately weathering rock.

Q original, say,	=	4.2
Wall adjustment	=	2.5 Q
Orientation adjustment	=	0.75 Q
Weathering adjustment	=	0.75 Q
Q_R=Q×2.5 ×0.75×0.75		
Q_R=1.4 Q=5.88		

• CRITICAL PARAMETERS IN ASSESSING RAISEBORE ROCK QUALITY

Parameters critical to raisebore stability are the intensity of jointing which defines block size (RQD/Jn) and interblock shear strength (Jr/Ja). Problems may be anticipated in rock masses with small to medium rock block size and low interblock shear strength. Table 1 indicates a raiseboring classification for block Problems may be expected in large diameter size and low interblock shear strength. raises if the critical parameter values are poor or worse.

TABLE 1 CLASSIFICATION OF CRITICAL GEOTECHNICAL PARAMETERS IN RAISEBORING

RAISEBORE CLASS		V. POOR	POOR	FAIR	GOOD	V. GOOD
RQD/Jn	2	–4	–8	–15	–25	–50
Jr/Ja	0.25	–0.5	–0.75	–2	–3	–4

RAISEBORE STABILITY RATIOS (RSR)

The Raisebore Stability Ratio, RSR, is equivalent to Barton's Excavation Support Ratio, ESR which is dependant on excavation function and life, and ranges from 0,8 for long term public facilities to 5 or more for temporary mine openings. Raisebore shafts generally have a medium to long term serviceable life requirement. An RSR of 1.3 has been assigned and is inherent in all the tables and charts which follow. This is considered commensurate for a ventilation shaft. For ore passes a higher figure, say 1,6, could be used.

RAISE STABILITY AND RAISE DIMENSIONS

The major factor in raisebore stability is the raisebore diameter. Raisebore length is a contributory factor in terms of incidence of sidewall failure but does not affect stability per se. The circular shape of the excavation is generally accepted as optimal in terms of stability in a uniform lateral stress environment. Wall stability is again controlled by the diameter of the shaft (analogous with tunnel wall stability being controlled by the height of a tunnel).

Face stability can be simply determined in terms of a maximum unsupported span where :

$$\text{span}_{max} = 2RSR\ Q^{0.4}$$

Figure 4 presents the relationship between maximum unsupported span and Q_R (at RSR=1, 3). It can be seen that the required Q_R value for stability increases rapidly with increasing raise diameter. For example a 4m raise will be marginally stable in poor quality rock, Q=2,8, whereas a 6m raise requires a fair quality rock with Q-8.

From this it is apparent how few problems were experienced in the past with small diameter raises. Raises of up to 4m in diameter would therefore experience only isolated incidents offailure given poor to fair quality ground.

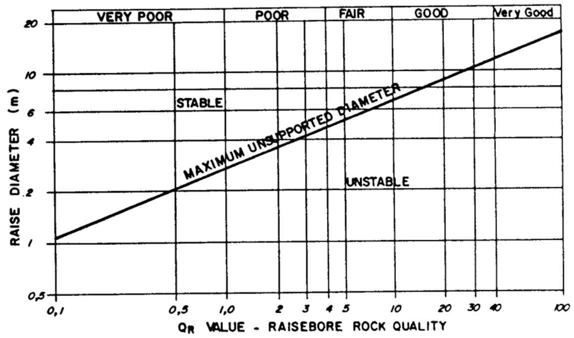

FIG. 4 RELATIONSHIP BETWEEN MAXIMUM UNSUPPORTED RAISE DIAMETER AND RAISEBORE ROCK QUALITY

During raising it is the face which is more liable to instability. During the cutting operation the raisebore head itself provides a degree of support due to the pressure of the cutters. Whenever the head is lowered this support is withdrawn. It is at this time when failures are most likely. Failures from the edges of the face can subsequently undercut the sidewall. An uneven face and especially an enlarged eccentric diameter is particularly difficuly to re-initialise the cutter head to recommence drilling.

This in turn creates vibrations in the head which are transmitted to the rock leading to further rock failures and induces stresses in the equipment.

GEOTECHNICAL VARIABILITY AND RISK ASSESSMENT

The determination of tunnelling or raisebore rock quality as described above is limited in that unique values are generally chosen as input into the Q equation. To be conservative, lower bound values of RQD, Jr and Jw and upper bound values of Jn, Ja and SRF may be chosen. This may often lead to unacceptable conditions being indicated. Clearly as raise diameters become larger it is more important to more accurately determine the rock mass quality or alternatively have a way of calculating the reliability that the quality is above a required minimum value.

The risk attached to any raisebore project will depend upon the confidence with which the relevant parameters are known. The level of confidence or reliability of information depends on :

- amount of information available
- variation of individual parameters
- impact of this variability on the probable tunnelling quality index
- required minimum rock quality for compatibility with the proposed raisebore shaft specifications

The important aspect is to assess the ground conditions with respect to the required minimum quality for stability considerations (as obtained from figure 4.) The distributions of each of the contributing parameters must be defined especially those with the greatest bear. ing on the shaft stability (ie joint shear strength and joint spacing).

Fig 5 presents a flowchart showing the activities to be followed for a systematic assessment of the risk related to geotechnical aspects of any raiseboring project.

In mining there will generally be a significant amount of geotechnical data available. This is not normally the case in a civil engineering project. In all cases there will however be a top (often the surface) and bottom station. These should be examined together with all other sources of data for input into a preliminary geotechnical evaluation and risk assessment.

Initial Risk Assessment

The preliminary geotechnical assessment should be aimed at determining average and lower bound conditions in terms of ‘raiseability’ and stability.

The range and distribution of the raisebore rock quality Q_R, and the important RQD/Jn and Jr/Ja parameters must be compared to the required minima for stability at the proposed shaft diameter.

In addition to simply assessing the range of predicted Q_R values against those required, the rockmass properties and discontinuity orientations would be used as input to detailed stability analysis. These would ideally be conducted on a probabilistic basis by sampling the distributions of the various properties and thereby determining the probabilities of failure in terms of both incidence of failure along the raise length and volumes of failure. The failure wedges would be analysed in

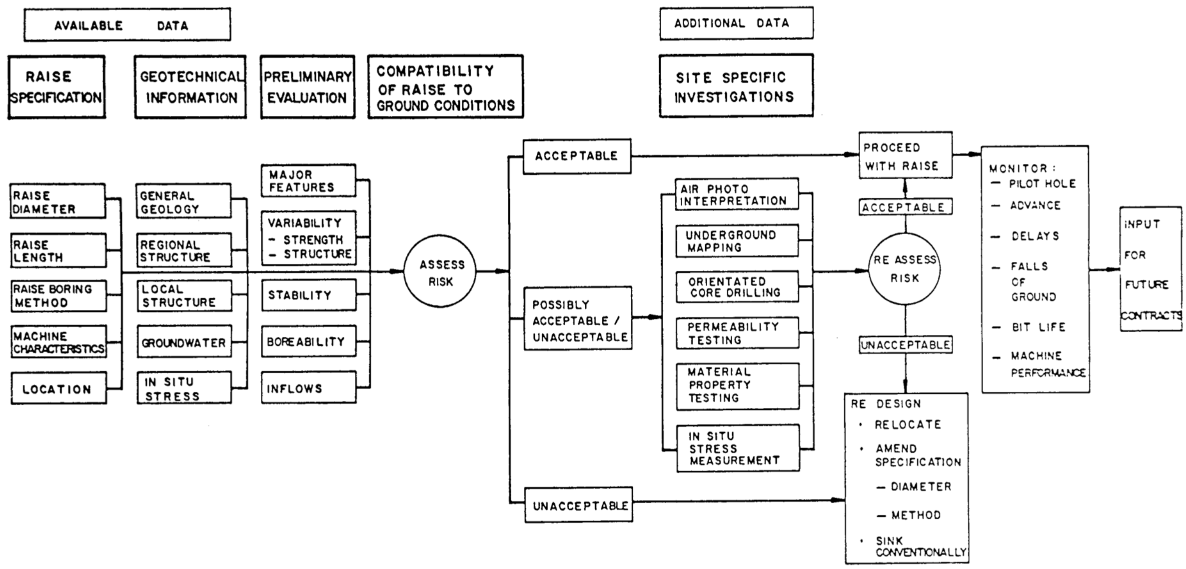

FIG. 5 FLOWCHART FOR GEOTECHNICAL RISK ASSESSMENT OF RAISEBORING

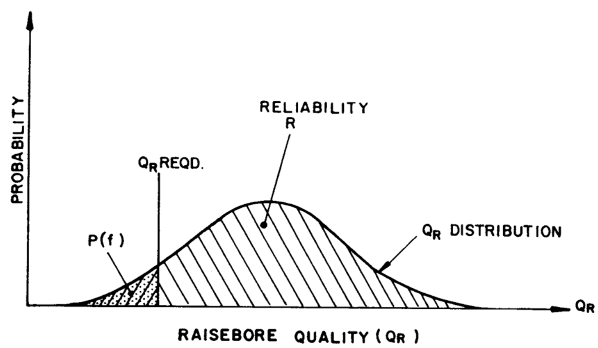

FIG. 6 RISK ASSESSMENT DIAGRAM

terms of the potential for 'keyblock failure', ie the failure of a particular block which would lead to a domino effect with subsequent gravity failures causing significant damage to the raise wall.

At the preliminary evaluation stage the risk should only be deemed 'acceptable' if the tunnelling quality is consistently indicated to significantly exceed (ie be in the next class up from), the required quality throughout its length. This presupposes sufficient information to draw this conclusion. Conversely, risk is deemed 'unacceptable' if the tunnelling quality is consistently indicated below that for the proposed shaft. Marginal cases occur where the indicated quality is not confidently known or where the distribution of Q_R straddles the required level. In these cases either additional information must be gained or alternatively the acceptability of the level of risk associated with the required QR value with respect to the defined QR distributions must be judged. The risk to the raise can be assessed from Fig. 6 which compares the required raisebore quality with the in situ rockmass quality. The area under the quality distribution curve to the left of the required quality line is where the under the complete curve equals unity and the stability of the shaft is at risk. The area shaded area to the right of the required quality represents the reliability where Q_R is greater than that required.

The method of obtaining the Q_R distribution is described by Harr[3]. Alternative probabilistic methods of determining reliability based on multivatiate point estimate methods are also described[3].

FINAL RISK ASSESSMENT

The final assessment of risk will ultimately depend on the acceptability of failures within the raisebored shaft and the incidence and volume of failures which can be tolerated.

In general an acceptable probability of failure of a raisebored shaft given its function is considered to be 0, 05, ie 5%. This is commensurate with an RSR of 1, 3. Given a proposed raisebore diameter and a rock mass of a certain range of raisebore quality, one can obtain the range of probability of failure. Knowing the length of the raise one can calculate the likely extent of raise wall which would be affected by failures with the volumes of failure being determined from stability analyses.

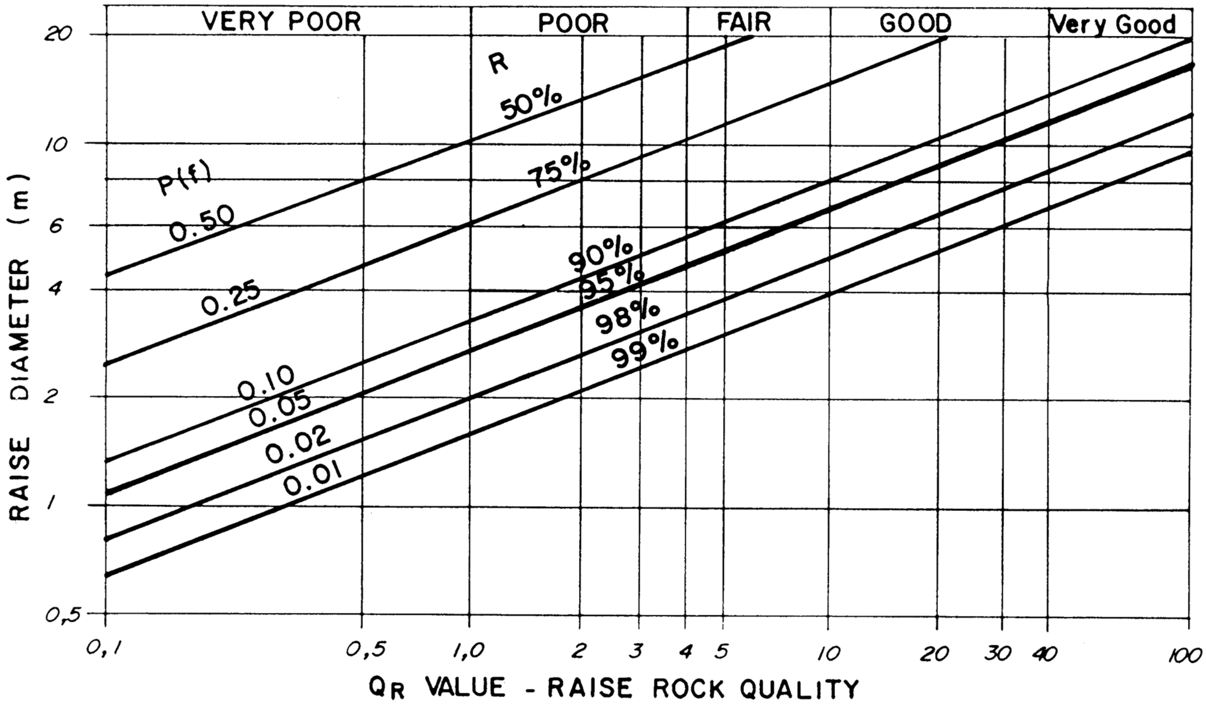

FIG. 7 RELIABILITY/PROBABILITY OF FAILURE CHART : RELATIONSHIP BETWEEN RAISEBORE QUALITY INDEX AND RAISE DIAMETER FORCE

Figure 7 presents the probabilities of failure (Pf) or alternatively the reliability (R) of a shaft (Reliability R=1−P(f)×100%) for the range of raisebore diameters and rock mass qualities.

• ACCEPTABILITY OF RISK

The acceptability of the reliability against instability is considered to be different for the raise face and the raise walls due to the following:

- the raise face is temporary; failure from the face is likely to affect the raise bore head more than it will affect the final shaft.
- the raise walls are permanent; instability of the walls may lead to progressive failure affecting the shaft after the completion of the raise.

It may be considered that a higher level of reliability of stability should be obtained for larger raise diameters as the consequences of failure increase with increasing diameter.

The life of the excavation should be taken into account (better reli-abilities being required for shafts with longer serviceable life). Also the function of the excavation should be considered. In mining, function and serviceable life at a shaft are often linked. eg. a ventilation shaft is generally a long term excavation critical to production and would require a higher level of reliability than an ore pass which is not equipped and generally of a shorter life span. This said however, the stability of the ore pass must be ensured over its production period.

Table 2 presents levels of reliability (R) and probabilities of failure (Pf) that are considered acceptable for raise wall stability for the various excavations. These provide guidelines for other raisebored shafts.

TABLE 2: ACCEPTABILITY OF RISK FOR VARIOUS RAISEBORED SHAFTS

Excavation	Acceptable Service Life	Risk R	P(f)
Unlined hoisting Shaft	+ 15 years	99%	0.01
Ventilation shaft	10 years	95%	0.05

Excavation	Acceptable Service Life	Risk R	P(f)
Ore pass	+ 2 years	85%	0.15
Ore pass	−1 year	75%	0.25

CONCLUSIONS

The remoteness of raiseboring requires that raisebore projects are approached more cautiously than conventionally sunk shafts.

A method of quantifying the geotechnical risk to a raisebore shaft is presented based on the shaft diameter and a raisebore rock quality index Q_R

The approach outlined provides and indication of overall geotechnical feasibility. All excavations must however be considered individually and the potential problems should be addressed on merit. The chart presented does not replace the necessity for classical analysis to evaluate the incidence and stability of potential failure wedges. It does however allow the probability of failure to be predicted in a simple manner. When compared to the required reliability, the overall feasibility and the risk to a proposed raise can be assessed.

REFERENCES

1 **Barton N, Lieu, R. and Lunde, J. (1974)** Engineering Classification of rock masses for the design of tunnel support. Rock Mechanics Vol 6 pp 189–236.

2 **Kirsten H A D (1983)** The Combined Q-NATM System—the design and specification of primary tunnel support. South African Tunnelling Vol.6 No. 1.

3 **Harr M E (1987)** Reliability Based Design in Civil Engineering, McGraw-Hill N.Y.

Performance observations on the pressure shaft for the Chhibro underground power house complex

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SYNOPSIS

The Chhibro underground power house project was completed 14 years ago. The complex is located in poor quality dolomitic limestones and construction was a challenging task since this was the first large cavern to be built in the tectonically active zone of the Himalayas. Instrumentation was installed during construction to monitor stress and strain conditions in the rock mass and the lining. Despite a significant number of instrument failures, useful data have been obtained on the post-construction and service behaviour of the excavations. This paper deals specifically with the analysis and interpretation of data collected over a three year period. The observations indicate that the water level in the surge tank and the water temperature influence the stresses in the steel liner of the pressure shaft although earthquake vibrations do not.

INTRODUCTION

Construction of large underground caverns for engineering purposes is becoming increasingly common as a result of demands arising from space constraints, strategic requirements, operational safety needs and security against damage by natural causes. The first underground power house was constructed in the USA in 1898–99. Over the last three decades, many underground power houses have been built and more are currently under construction. In this context, the Chhibro underground power house complex has set a major precedent by being the first venture of its type in the Himalayas.

The complex is located in poor quality dolomitic limestones which include numerous shears, faults and other discontinuities. The machine hall has a semicircular crown and is 113.2 m long×18.4 m wide× 32.5 m high. The surge shaft complex has a 20 m diameter underground surge tank of the restricted orifice type. This has a height of about 95 m and is sited in limestones and slates. The top of the surge tank is connected to a 7 m diameter upper expansion gallery and ventilation tunnel and a 16 m diameter riser which joins to the headrace tunnel.

There are four pressure shafts feeding the four turbines of the power house. All the shafts are located in limestone which is thinly bedded, interlayered with thin slate bands and intensely jointed. The limestone also includes thinly bedded shear zones mostly at the limestone-slate contacts. The shafts are all 3.8 m internal diameter and steel lined. Lining thickness varies from 14 mm near the surge tank to 36 mm near the power house. Shafts 1 and 4 are about 120 m long whereas the other two are about 110 m long.

The shaft steel liners are backfilled with M-20 concrete with a minimum thickness of 600 mm. The steel liners are designed for internal hydrostatic pressures as well as the external pressures which may arise either during grouting or when the shafts are dewatered. The stiffening rings for the circular liners consist of 150×150×20 mm angle iron welded with its apex upwards. The space between the liner plate and the angles is filled with cement slurry. Grouting was used to fill the gaps between the concrete lining and the rock, the steel liner and the concrete, and for consolidating the surrounding rock. Grout holes were provided at about 2.5 m centres along the length of the liner shells. Four holes were provided in each ring, equally spaced around the circumference and with the holes staggered in adjacent rings.

The planning, design and construction of this underground power house was done with very limited data on the rock mass and the cavern was completed with considerable difficulty. However, the power plant has now been functioning without incident for more than fourteen years.

During construction, a network of instruments (e.g. extensometers, rock bolt load cells, strain meters and piezometers) was installed in the power house complex to check the safety and adequacy of the support systems, to assist with the assessment of any required remedial measures and to monitor the post-construction behaviour of the excavation.

This paper discusses the analysis and interpretation of instrumentation data over a three year period. Water level and temperature variations are shown to affect the stresses in the steel liner of the pressure shaft whereas earthquake vibrations appear to have little effect.

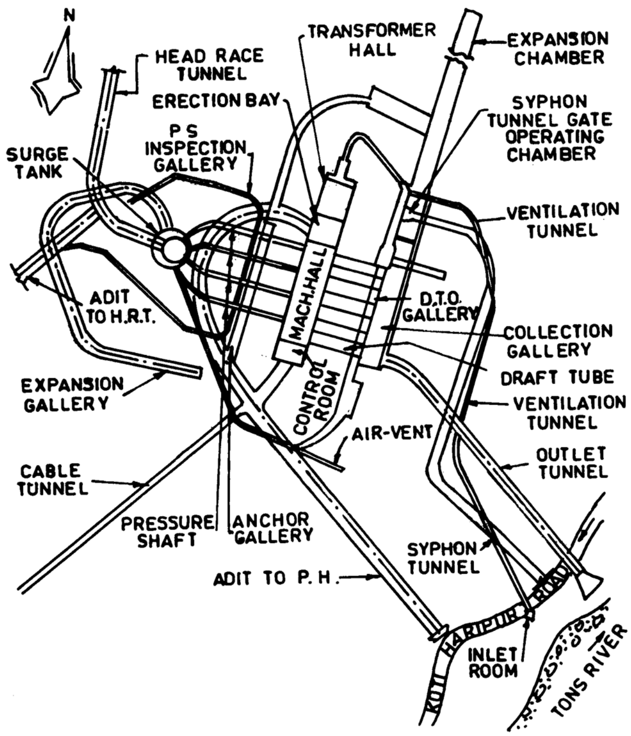

Figure 1 General layout of underground powerhouse

PROJECT DESCRIPTION

The Yamuna Valley Development project started in 1949 when the foundation stone of the Stage I works at Dakpathar was laid. Thereafter, several other stages were investigated and are currently being executed.

The Yamuna Hydroelectric Scheme Stage II involves the development of the power potential of the Tons River, a tributary of the Yamuna, between Ichari and its outfall at Dakpathar in Dehradun district, U.P. The total head available is about 188 m which is being utilised in two stages.

Part I uses the 240 MW Chhibro power station to exploit the drop of about 124 m along the first loop between Ichari and Chhibro. This was the first venture of its type in the Himalayas and was necessitated because the location of a surface power station would have involved large scale excavation of steep slopes. Extensive geological investigations were undertaken to optimise the location of the underground power station. The final decision involved the siting of the power house complex in a band of limestone which has a horizontal width of 193 to 217 m. The Chhibro complex comprises a network of excavations for the machines, transformers, turbine inlet valves and control room and also provides operating galleries and hydraulic connections to the Part II stage. This latter stage involves a 120 MW scheme utilising the remaining drop of 64 m along the second loop between Chhibro and Dakpathar. A general layout of the Chhibro underground power house complex is shown in Figure 1.

Part I of the Yamuna Hydroelectric Scheme Stage II commenced in March 1966 and became operational in March 1975. The main components of this project are:

(i) 60 m high concrete diversion dam across the Tons River at Ichari together with spillway, intake sedimentation chambers, flushing arrangement and tunnel control structure.
(ii) 6.3 km long by 7.3 m finished diameter, concrete-lined headrace tunnel with design discharge capacity of 235 cumecs.
(iii) 20 m diameter by 100 m high underground surge tank at the end of the headrace tunnel
(iv) 4 no. steel-lined, 3.8 m diameter, pressure shafts feeding the 4 no. turbines.
(v) Underground power house at Chhibro with four machines of 60 MW each.
(vi) Tailrace works comprising collection gallery, expansion chamber, outlet tunnel, outlet structure and a syphon tunnel interlinking the tailrace works of Part I with the headrace tunnel of Part II.

Because of the 6.3 km length of the headrace tunnel, it was necessary to provide a surge shaft upstream of the power house to serve the following functions :

(i) To provide a free reservoir surface close to the machines as a rapid means of compensating for water hammer effects, thereby limiting penstock pressures and materially reducing tunnel pressures.
(ii) To supply the additional water required by the turbines under increasing load demand until the conduit velocity has accelerated to a steady state value.
(iii) To store water during load rejection until the tunnel water velocity has decelerated to a steady state value.
(iv) To ensure that oscillations of water level following load changes are rapidly damped.

GEOLOGICAL SETTING

The Tertiary rocks of the Lower Himalayas are young rock formations. These formations are inherently weak and are further complicated due to the active nature of the Himalayas. The geology is complex and the rocks are intensely folded, faulted and sheared. The lithological conditions and tectonic complexities coupled with the rugged and inhospitable terrain precluded thorough geotechnical investigations being carried out prior to construction.

The Palaeozoic formations comprising interbedded quartzites, slates and limestones are exposed in the power house complex excavations. The rocks have a varying dip ranging from 30° to 80° in the directions of north, north-east to north-north-west. The surface exposures are thickly bedded to massive but appear broken and shattered because of slump cracks. Bedding plane partings are present in the slates although these are generally tight.

The power house excavations are located in a band of stratified limestone of 25 m true thickness (200 m across in the horizontal direction) with minor or thinly bedded slate bands. The rock is closely jointed with numerous shear zones ranging from 20 to 500 mm thick and which are sub-parallel to bedding. A major shear zone approaches to within 10 m of the lowest draft tube level in the power house area. The formations dip at about 45° towards N150°W to N29°E. The excavation is aligned normal to the strike of the rock formations with cover ranging from 208 m over the control room to more than 250 m over the transformer hall (Gupta et al, 1985).

Despite the problems mentioned above, the 113 m long×18.5 m wide×32 m high cavern was successfully constructed. Construction took about eight years and many unique problems were encountered. Since this was the first such venture in Himalayan rocks, no prior experience or case records were available for reference. The power house has now been functioning for over fourteen years without any stability problems. Figure 2 shows the geological cross section and instrumentation layout.

FIELD INSTRUMENTATION

It was not possible to undertake thorough geotechnical investigations for the reasons mentioned above. A geological cross section was prepared by projecting the geological features exposed at the surface. Exploratory drifts indicated poor quality rock mass. The in-situ deformation modulus of the rock mass was determined by tests carried out in drifts and was found to be in the range of 1.36 to 1.93×10^4 kg/cm^2. Measurements of the in-situ state of stress showed that conditions were nearly hydrostatic with the vertical stress being comparable with the overburden pressure (Dube and Singh, 1986). Since there were only very limited data on the rock characteristics in the area, design work was largely based on rock response measurements using locally made devices. Because of the lack of precedence for this work, there was considerable interest in monitoring the post-construction behaviour of the power house complex. Longterm instrumentation was therefore installed to observe the behaviour of the underground complex so that design assumptions could be checked against the results of the ongoing observations and timely action could be taken where problems were indicated. Apart from the safety of the works, the data so obtained will help in assuring better economy and greater stability in the design of future underground structures in the Himalayas (Mitra et al, 1988).

Although only limited instrumentation could be carried out, three arrays of strain meters were installed and monitored in the steel liner of the pressure shaft. A cross section of the shaft and details of the strain meter installations are shown in Figures 3 and 4. The strain meters were of the vibrating wire type since these have the best longterm performance. The instruments were of the vibrating wire type and were read by remote readout. A detailed analysis of the results is given in the following sections on data analysis and discussion.

ANALYSIS AND DISCUSSION

The present paper discusses the hoop stresses in the steel liner of the pressure shaft. Four sets of three strain meters were installed at the penstock limbs. These recorded the induced strains in the liner and hence allowed the stresses to be calculated. The results obtained for 1977–1980 are shown on Figures 5 and 6 where the three instruments indicated are at the same elevation in the same shaft liner. Although the absolute stresses measured by these instruments vary considerably, it can be seen that the stress changes are very consistent from instrument to instrument.

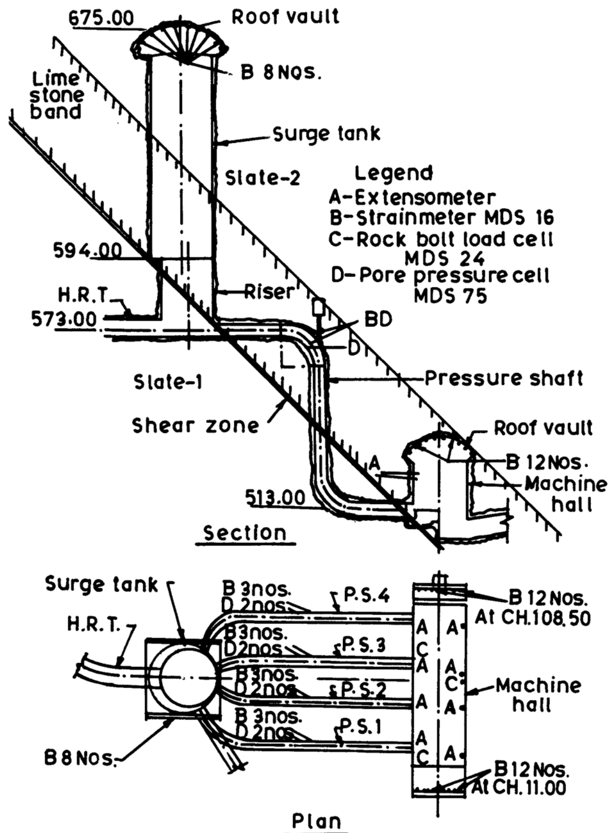

Figure 2 Geological cross section and plan with instrumentation layout

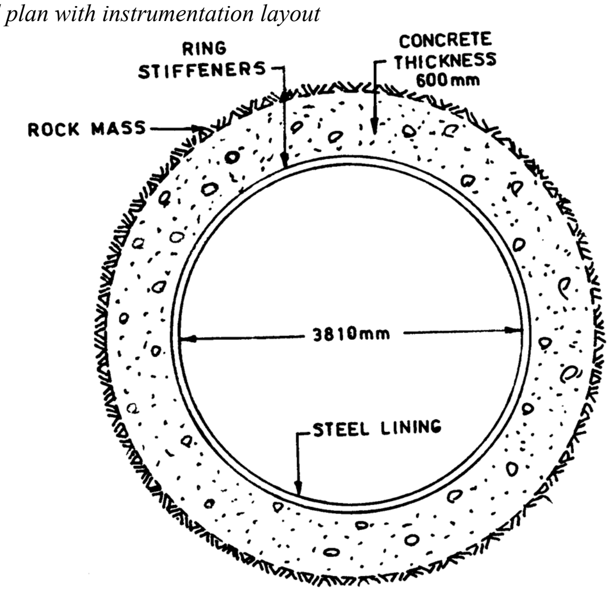

Figure 3 Cross section of pressure shaft

A seismological observatory is being maintained to record earthquake shocks in the area. This is situated at the left bank of the River Tons about 10 km upstream of the power house complex. Regular records were also kept of other important data, such as water level in surge tank, rainfall, pore water pressure around pressure shaft, temperature of water and atmosphere, quantity of water drawn for power generation. Typical variations in liner stress with time are shown on the above figures. The date of observation is shown along the horizontal axis; the vertical axes show the observed stress in N/mm^2, level of water in the surge tank, calculated thermal stresses in N/mm^2 and earthquake vibrations in terms of number of shocks for epicentres within 110 km range. Thermal stresses have been calculated taking into account the interaction between the steel liner and the rock mass (Kumar, 1988). The concrete has been assumed to be cracked and therefore its interaction with the steel liner or the rock

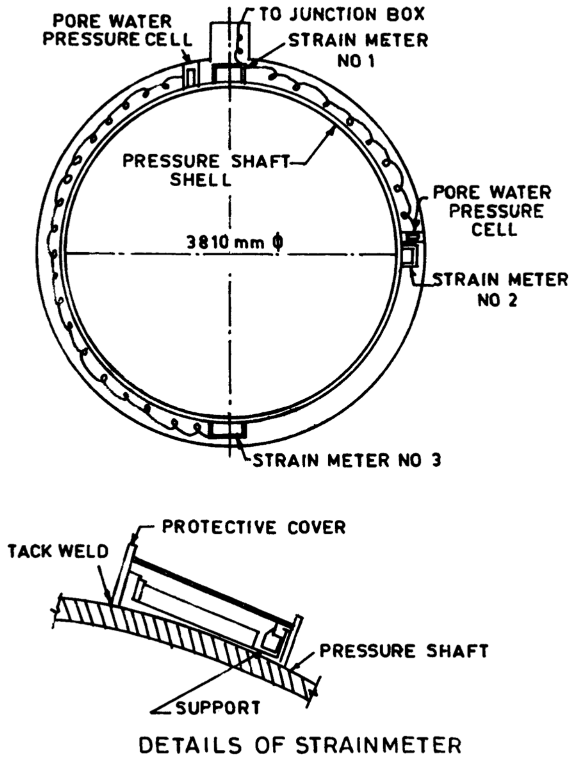

Figure 4 Strain meter in steel liner

mass has been ignored in calculating thermal stresses. The present authors believe that a complete equation should be derived taking into account the interaction between the steel liner, the concrete and the rock mass. The common equation for calculating stress increases in linings with changes in water temperature is **[EaT/I−v]** where **E** is the modulus of elasticity of steel, **a** is the coefficient of thermal expansion of the steel, **T** is the temperature difference between two readings and **v** is the value of Poisson's ratio for the rock mass. The observed stress changes in the liner with increase in water temperature were found to be much less than the values calculated on the above basis.

It is clear from the figures that the liner stresses are affected by the thermal stress pattern and the water level in the surge tank. Outside of the monsoon season, the water level of the reservoir/surge tank is kept high and the discharge in the tunnel/ pressure shaft is relatively low. Hence the internal water pressure during this time is always greater than that in the monsoon period (Goyal and Rajvanshi, 1978). Similarly the temperature of the water (which varies with the season) flowing through the pressure shaft has been used to calculate the thermal stresses induced in the steel liner. Atmospheric temperature variations have not been considered in calculating thermal stresses because the temperature in the underground power house is more or less constant. The installed instrumentation did not include any provision for dynamic recording of stresses during earthquakes. However, Figures 5 and 6 show that the stresses in the liner do not increase with time as would be the case if strain energy was accumulated after earthquake shocks. It is concluded that earthquake vibrations do not influence the stresses in the steel liner. By comparison, the anchor loads in the machine hall have been found to increase slightly during the rainy season and during earthquakes.

CONCLUSIONS

1. The stresses in the steel liner are influenced by the water level in the surge tank.
2. The liner stresses vary significantly with the temperature of the flowing water.
3. Earthquake vibrations do not appear to affect the liner stresses.
4. Anchor loads in the machine hall have been observed to increase slightly during the rainy season and during earthquakes. This effect has not been observed in the shaft liner since the shaft is inherently more stable than high caverns.

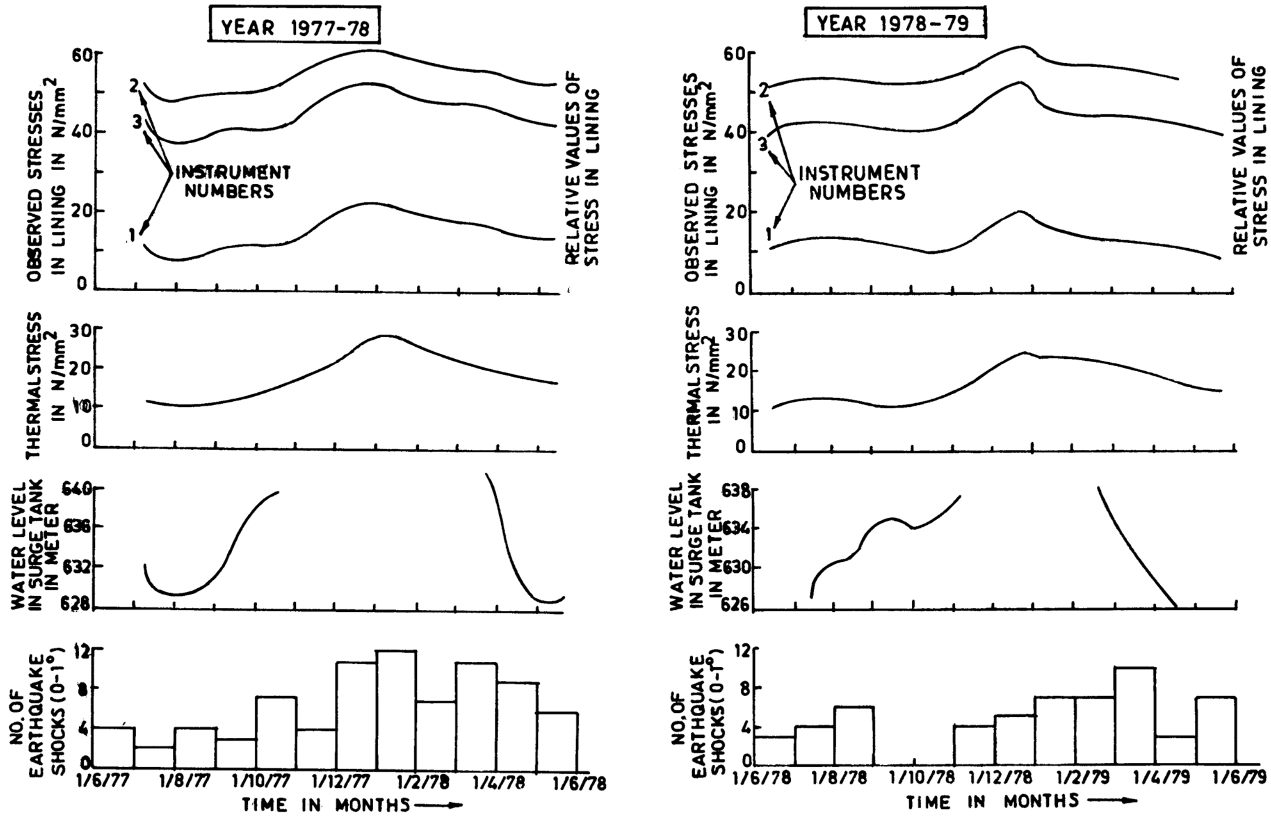

Figure 5 Variation of stresses in steel liner, thermal stresses, level of water in surge tank and earthquake shock with time (1977–79)

ACKNOWLEDGEMENT

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REFERENCES

Dube A.K. and Singh B. Post construction behaviour of a large cavern in poor rocks of Lower Himalayas. Proceedings of International Symposium on Large Rock Caverns. Finland, 1986, Vol II, pp 943–946

Goyal K.G. and Rajvanshi U.S. Instrumentation of works of Yamuna Hydel Scheme Stage-II Part-I and their performance. Proceedings of All India Symposium on the Economic and Civil Engineering Aspects of Hydroelectric Schemes. Roorkee, 1978, pp VIII-65 to VIII-74

Gupta J.P., Agarwal D.K. and Jain K.K. Construction of Underground surge shaft at Chhibro. Proceedings of Third Symposium on Rock Mechanics. Roorkee, 1985, pp 92–100

Kumar P. Development and application of infinite elements of openings in rock mass. Ph D thesis. Dept. of Civil Engineering. University of Roorkee. 1988

Mitra, Subhash, Singh, Bhawani and Rajvanshi U.S. Performance of Chhibro underground powerhouse cavities after charging water conductor system. A case study. Proceedings of International Symposium on Underground Engineering. New Delhi, 1988, pp 425–430

Mitra, Subhash and Singh, Bhawani. Behaviour of support system of large underground openings during earthquake vibrations. Proceedings of Indian Geotechnical Conference. Allahabad, 1988 (in press)

Terzaghi K. Theoretical soil mechanics. John Wiley & Sons, 1943

Yearly reports on data collection and analysis of Yamuna hydel scheme, Stage-II, Part-I works, 1977–1980

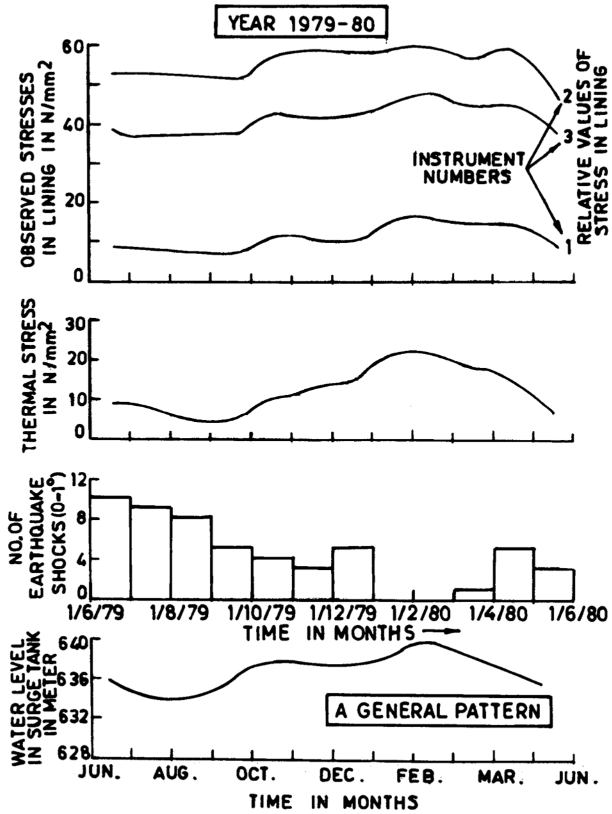

Figure 6 Variation of stresses in steel liner, thermal stresses and earthquake shocks with time (1979–80) and general pattern of water level in surge tank

Shaft sinking at Sikfors power station

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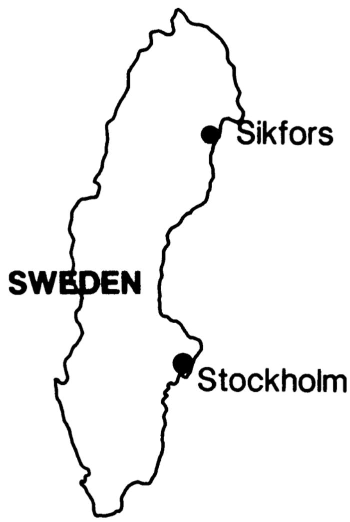

FIG 1. LOCATION OF SIKFORS POWER STATION.

SUMMARY

In the northern part of Sweden, Skanska AB and their subsidiary company Stabilator successfully constructed a shaft with a depth of 19 m. The shaft had an external diameter of 23.4 m. A caisson of reinforced concrete was constructed at ground level 20 m from the Piteå River. From that level the caisson was excavated down to the bedrock through 19 m of very dense silty moraine containing a large number of boulders.

A groundwater lowering system was installed including 40 no. filter tubes and a number of piezometers. The groundwater level was then successfully reduced by 15 m.

The maximum skin friction was estimated to exceed 5,000 tonnes. As the weight of the caisson was approx. 3,000 tonnes, it was planned to use a system of hydraulic jacks and rock anchors to overcome skin friction and point resistance. The actual skin friction turned out to be low since a gap was established around the upper part of the caisson. During the sinking phase the jacks were basically used to keep the caisson upright.

During the final phase of the shaft sinking, it became difficult to maintain the caisson in a vertical position. The groundwater level was slightly higher on one side of the caisson and the strength of the soil material very low with the result that the caisson started to lean towards this side. By means of backfilling and using the jacks to apply pressure on one side, the caisson was successfully returned to an upright position and then sunk down to the bedrock.

INTRODUCTION

In connection with the extension of Sikfors power station a new tailrace structure was constructed. Fig. 1 shows the power station location.

A vertical shaft was included as part of the tailrace structure. A caisson of reinforced concrete with a height of 18.65 m and an outer diameter of 23.40 m was constructed at ground level and then excavated downwards to the bedrock (Fig. 2). The shaft was placed at a distance of about 20 m from the Piteå River, which at high water has a flow rate of 400 m3/s.

A temporary cofferdam was built to protect the site against the river.

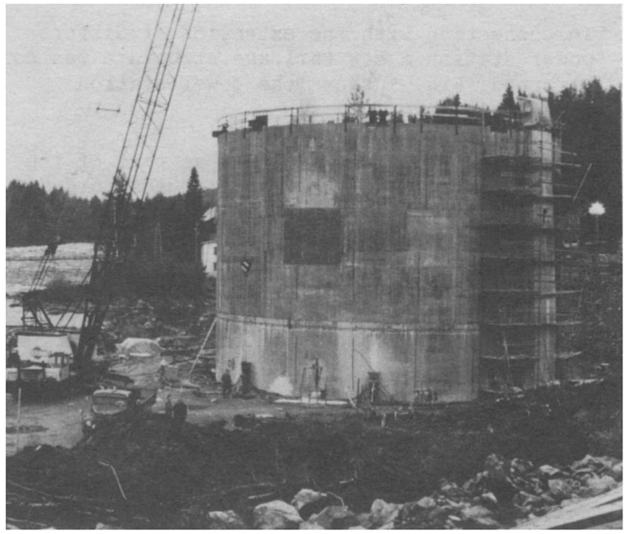

FIG 2. THE CAISSON AT GROUND LEVEL.

GEOTECHNICAL CONSIDERATIONS

The caisson was to be excavated through 20 m of soil down to the bedrock. The soil material was classified as a very dense silty moraine. Horizontal layers of sand, gravel and boulders of up to 2–3 m^3 in volume were anticipated in the material. The silty soil was liable to flow if encountered below the water table. The permeability of the material was estimated to be 10–6 m/s.

Below the soil there was a layer of 2–3 m of fissured rock with clay-filled cracks. Contradictory results were obtained when probe drilling to determine the location of sound rock.

Probe drilling was performed in two parts with different people and equipment. The criterion for sound rock was obviously not the same for the different people.

PROPOSED METHOD OF SHAFT SINKING

Based on previous experience from similar works the expected skin friction was estimated to be in the order of 3,300–5,000 tonnes. A wide scatter of results was obtained when different experience and calculation methods were applied. To be able to sink the caisson with a dead weight of about 3,000 tonnes it was anticipated that additional pull down force in the order of 2,000 tonnes would be required.

It was decided to provide the necessary pull down force by mounting 10 hydraulic jacks on the outside of the caisson. To counteract the jacks, rock anchors would be drilled down into the bedrock and grouted there. Prestressed bars would then be mounted between the jacks and the upper edge of the caisson.

Besides providing pull down force, the jacks would ensure the vertical position of the caisson, especially during the initial sinking phase until the surrounding soil could provide the necessary guidance.

Pipes would be installed in the lower part of the caisson to enable bentonite lubrication to be applied in case the skin friction and point resistance were observed to be too great.

The aim was to lower the groundwater level substantially. Calculations showed that lowering the groundwater to a level about 5 m above the rock surface was feasible. If there was a possibility of lowering the groundwater level further, the necessary measures would be taken. However, preparations were made for freezing in connection with the last 5 m of the shaft sinking.

LOWERING OF THE GROUNDWATER LEVEL

Initially the groundwater level was about 1 m below the ground surface.

Forty filter tubes placed in two circles were installed to effect the lowering of the groundwater level. (Fig. 3.)

In the inner circle 18 filter tubes were installed primarily to enable sinking of the caisson in dry conditions. The remaining 22 filter tubes were placed in an outer circle to enable supplementary excavation and concrete work on the outside of the caisson to be carried out at a later stage.

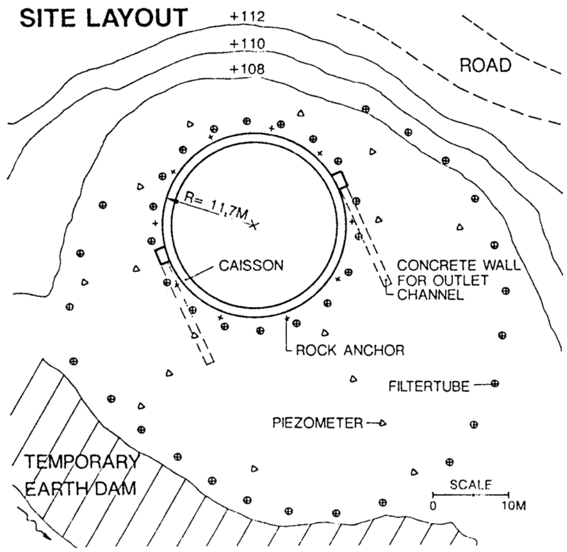

FIG 3. LAYOUT OF SITE FOR SHAFT SINKING.

During test pumping a very slight flow of about 0.2–1.0 m^3/h was noticed. Artesian water was also observed in the underlying rock. The filter tubes were designed with perforated plastic pipes and were drilled 3 m down into the sound rock. A layer of washed gravel was placed around the outside of the plastic pipes.

The water level was checked daily by means of 16 piezometers placed within the working area. When all the filter tubes were installed, pumping was started simultaneously. The groundwater level was then lowered 13 m in 4–5 weeks. Slight variations occurred within the area, but in principle the groundwater level was 6 m above lowest level planned for the shaft.

To lower the groundwater level further, vacuum pumping was tried in four of the filter tubes. Vacuum pumping gave effect locally by lowering the level a further 2–3 m. This was not considered sufficient to justify a complete installation for vacuum pumping of all filter tubes.

A computer-based calculation was performed, on the basis of results reported from the actual pumping operations, to find out why it was impossible to lower the groundwater level down to the bedrock. The calculation showed that a vertical inflow of water from the rock would occur within the area. The fact that artesian water had been observed earlier also supported this conclusion.

Five deep wells were then drilled down to a diabase dike that lay 50 m below the rock surface at the shaft. Although a flow of 200–500 1/min was pumped from these wells, no lowering of the groundwater level was observed.

The reason why it was not possible to lower the groundwater level down to the bedrock later proved to be that horizontal sand layers in the soil transported water between the filter tubes although they were placed on centres of only 4.0 m. Another significant factor was that a number of filter tubes gradually filled up with sludge and in some cases were not functioning at all.

Despite regular flushing and the execution of supplementary filter tubes, it proved to be impossible to lower the groundwater below a level 2–3 m above the bedrock.

The idea of carrying out the shaft sinking and connecting works under completely dry conditions was now abandoned and plans were made for the execution of part of the works below the water table. As the inflow of water was slight, this was judged to be feasible without having to resort to freezing.

SINKING OF THE SHAFT

In the initial phase the concrete caisson stood on a temporary support consisting of a circular beam of unreinforced concrete above a gravel bed.

Once the circular beam had been removed sector by sector, excavation started under the tip of the caisson. After one week when the caisson had sunk 2.30 m, the sinking operation had to be stopped due to the development of severe cracking in the lower parts of the caisson.

The first cracks were already discovered after the casting of the caisson but they were then only 0.3–0.5 mm in width. When the shaft sinking was stopped their size had increased to 5–7 mm. It was quite clear that the design of the caisson was at fault, and strengthening and repair works were started.

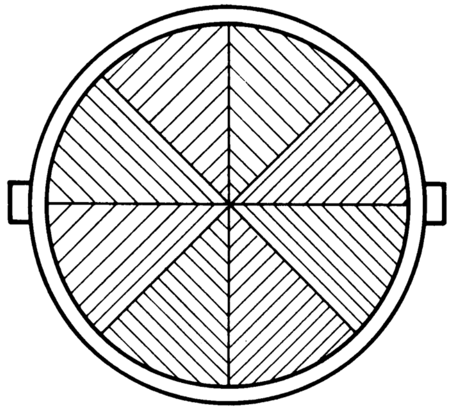

FIG 4. EXCAVATION PATTERN BENEATH CAISSON

The strengthening works consisted of the post-tensioning of 64 unbounded Dyform strands round the caisson. Observed cracks were injected with epoxy. The strands were pretensioned to 200 kN each.

The strands were covered with a 6–10 cm thick layer of shotcrete as a protection against mechanical damage during the sinking. Further steps were also taken to enable bentonite lubrication to be done.

The plan was to divide the periphery of the caisson into 10 sectors and then excavate under the caisson tip at every other sector. (Fig. 4.)

Thus excavation would be done under one half of the caisson tip. When this excavation was completed the force in the jacks would be increased successively to a maximum of 1,000 kN/jack.

With this procedure the caisson was sunk 3.0 m.

The dense moraine with its boulder content required more excavation to enable continued sinking.

Firstly it was necessary to excavate under half of the tip at all sectors and secondly to excavate under the whole tip at every other sector. In practice, excavation was also done slightly beyond the periphery on the outside of the caisson. (Fig. 5.)

Although the caisson was now standing on only five soil pillars, it still did not move. To start the caisson moving downwards, it was necessary to use the jacks to provide some initial movement.

This procedure was successful and resulted in an average sinking rate of the caisson of 3 m/week.

Excavation under the tip of the caisson was, as described, necessary to reduce resistance and to locate any boulders that might damage the caisson. Such excavation simultaneously caused a gap to form round the outside of the caisson. The degree of compaction of the soil was such that a vertical wall of soil up to 7 m high was formed outside the caisson. The gap finally became a crater extending outside the inner circle of filter tubes.

This resulted in the filter tubes becoming bent towards the caisson. The flushing of sludge which was done at regular intervals then became more difficult or impossible to execute.

When the caisson reached a depth of about 16 m, it began to incline more and more towards the side that was about 400 tonnes heavier. The concrete wall was somewhat thicker on this side and this created some eccentricity in the caisson from the beginning.

In addition the groundwater level on this side was somewhat higher and there were fewer boulders in the material. The soil resistance was therefore considerably less here than on the opposite side where the groundwater level was lower.

It was planned that if the inclination of the caisson exceeded 0.5° (1:100) it would have to be restored to an upright position. During the first 15 m of the sinking, the permitted inclination was maintained, mainly by occasional use of the hydraulic jacks. (Fig. 6.)

Attempts to set the caisson upright by tensioning the jacks and additional excavating on the opposite side were not successful any longer and the inclination gradually increased to 1.0° (1:50).

To restore the caisson to an upright position, it now became necessary to put cross beams under the tip on the lower side. It was then possible to set the caisson upright by excavating and using the jacks. Lubrication with bentonite was also applied at this stage, without apparent result.

FINAL PHASE OF THE SINKING OPERATION

The final 3 m of the shaft sinking proved to be difficult. As mentioned previously, the attempt to lower the groundwater level to the bedrock was not completely successful, and the last part of the shaft sinking had to be done in partially wet soil. The inflow of water into the caisson was controlled by pumps placed on the inside of the caisson.

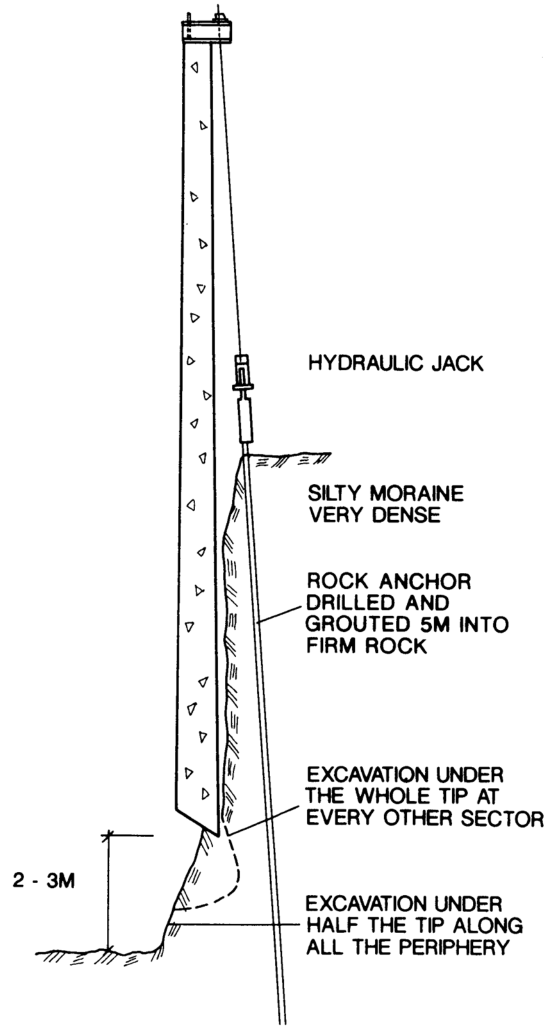

FIG 5. SECTION THROUGH CAISSON WALL DURING SINKING.

The crater formed round the shaft caused more and more of the filter tubes to stop functioning. The crater also enabled rainwater to find its way directly to the bottom of the shaft, causing local slides. (Fig. 7.)

An unexpected temporary failure of the power supply to the pumps had immediate consequences for the shaft sinking. The caisson sank 36 cm in an uncontrolled manner and the inclination increased to 1.5° (1:26).

To prevent loose masses falling into the crater and to stabilise the caisson, backfilling with crushed rock was done under its tip. Cementbentonite and polyurethane grouting was performed on the outside of the caisson to prevent inflow of water.

Finally the caisson was set upright. This was made possible at this stage by the close vicinity of the rock. Steel beams were now placed between the rock surface and the lowest section of the caisson. The caisson was then set upright again by simultaneously excavating and utilizing the jacks on the higher side.

Probe drilling was now done to determine the distance to sound rock. The surface of the rock proved to be very uneven around the periphery of the caisson. The distance to sound rock according to this additional investigation varied between 0.5 m and 3.0 m.

Sinking to sound rock continued by means of mechanical crushing and blasting of the remaining fissured rock. The mechanical crushing was done by pneumatic breakers.

The caisson was now temporarily secured by means of 8 steel beams placed between the caisson and the sound rock at those sections where the caisson did not reach the rock surface.

To stop the inflow of water between the tip of the caisson and the rock surface, additional, comprehensive grouting was now carried out. The whole area of sound rock inside the cylinder was cleaned carefully. A permanent beam of reinforced concrete was then constructed between the caisson and the rock surface.

No apparent inflow of water was observed from the rock surface although cracks of up to 3 cm in width were present.

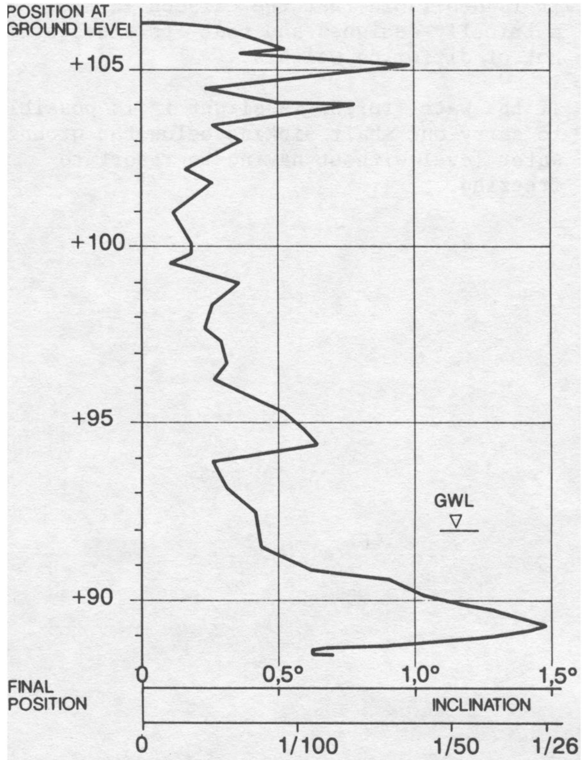

FIG 6. CAISSON INCLINATION AT DIFFERENT STAGES OF SINKING.

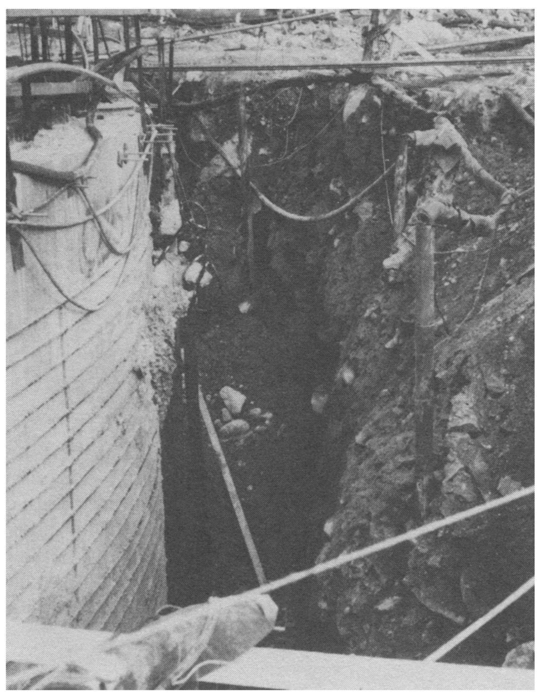

FIG 7. A CRATER FORMED ROUND THE SHAFT

CONCLUSIONS

Shaft sinking in a dense silty moraine makes high demands on the flexibility and adaptabil ity of the planned solutions. The theoretical assessments that can be made regarding the lowering of the groundwater level and skin friction before the actual start of the work are useful for preliminary guidance but have limited accuracy.

Experience from Sikfors shows that with flexible methods and good planning it is still possible to perform shaft sinking in such ground conditions.

When planning a similar project, special attention should be paid to the following points:

- Dry excavation is desirable for location of stones and boulders beneath the tip of the caisson.
- It is important that the level of the groundwater is nearly the same all the way round the periphery. The compressive strength of the soil on which the caisson rests will then be more uniform under the tip.
- When filter tubes are placed in an area where a crater may be formed, these must be equipped with stiff casing pipes.
- Lowering the groundwater to the level of the bedrock is difficult to achieve even if the distance between the filter tubes is small.
- Inefficient functioning of individual filter tubes may affect the level of the ground-water within a wider area.
- Hydraulic jacks are important and necessary for vertical steering.
- It is desirable that the caisson is symmetrically designed and that its sides are not of differing weights.
- If the water inflow is slight it is possible to carry out shaft sinking below the ground-water level without having to resort to freezing.

Design, construction and performance considerations for shafts for high-level radioactive waste repositories

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SUMMARY

Deep geologic disposal remains a preferred option for the permanent disposal of high level radioactive wastes produced by nuclear power generation. Site characterization for such a geologic repository will be conducted in part from underground excavations at the site in the exploratory shaft facility (ESF). Exploratory shafts will provide access to the potential repository horizon and will allow underground construction for site characterization and testing. These exploratory shafts are likely to become part of an eventual repository access system if the site is deemed an acceptable repository location.

Shafts provide the most obvious and visible connection between a repository and the biosphere, whether directly to the atmosphere, or to overlying or underlying groundwater or air within the geological formations adjacent to the repository. The main long-term health and safety hazard associated with High-Level Waste (HLW) repositories is the premature and excessive release of radionuclides to the environment. Given that the shafts could form one of the more direct pathways. and will be among the earliest repository structures to be developed, it is necessary that a conservative approach be implemented to preclude shafts from becoming preferential radionuclide migration paths.

Shaft design and construction requirements are governed by performance analyses. which must provide reasonable assurance that shafts will not allow excessive

The views expressed in this paper are those of the authors, and do not constitute official U.S. Nuclear Regulatory Commission positions. radionuclide releases. The regulatory requirements result in the need for careful control of numerous aspects of shaft design and construction. Technical items that are of particular concern in this regard are listed below:

(1) Shaft location: Shaft location aspects that need to be considered include potential for flooding and erosion, proximity to faults. proximity to areas designated for underground testing or waste emplacement, as well as risk of human intrusion (e.g. as affected by remoteness or accessibility).

(2) Excavation techniques: Shaft excavation techniques raise concern about the potential for creating an enhanced permeability zone adjacent to the shaft(s). Design of controlled blasting should be considered when conventional shaft sinking is used, to minimize the formation of preferential pathways along the shaft and consequently minimize the need for remedial sealing (such as grouting) of the rock mass.

(3) Rock mass characterization: Shafts will provide a major opportunity to characterize in detail. even if only locally. the rock mass between the repository horizon and the surface. This opportunity should be utilized to the fullest. i.e. construction operations should be compatible with extensive site data gathering requirements.

(4) Adverse impacts due to site characterization: The potential impact on long-term waste isolation of any characterization efforts. e.g. holes drilled from the shaft(s). will need to be evaluated in order to assure that site characterization does not have an unacceptable detrimental effect on the site isolation capability.

(5) Shaft reinforcement and support: ground control measures implemented during construction e.g. for conventional mining safety, affect shaft wall deformations and hence the development of a modified permeability zone near the shaft. Permanent shaft structures. systems and components should be designed and constructed for maintainable 100-years design life to provide for waste retrieval options. The shafts should be designed to perform under adverse repository-induced thermomechanical conditions. Reinforcement and support systems could affect permanent closure of the shafts upon completion of waste emplacement operations, and could affect long-term isolation capabilities of the site. depending upon the mode of deterioration of such components. Concrete and steel decomposition. for example. may result in chemical alterations affecting seal and rock hydraulic conductivity.

In sum, long-term waste isolation needs intrinsic to HLW respositories require the consideration of various shaft design and construction aspects from a perspective quite different from that of conventional shaft sinking. These unusual and rigorous requirements will affect all phases of the life of the shafts, from initial siting through permanent closure.

The present paper addresses shaft related technical concerns primarily from a site characterization point of view. Exploratory shafts will be sunk for site characterization, and are likely to become integrated components of an eventual repository. For this reason. they raise regulatory concerns. They will be among the earliest repository access routes to be developed. will be permanent. and will affect repository performance. Because of the potential repository performance implications, many regulatory concerns are well beyond and outside the scope of conventional shaft performance requirements, and hence shaft design bases. This paper intends to give a brief introduction and overview of such types of requirements, i.e. with emphasis on concerns related to HLW repository performance aspects.

1.0 INTRODUCTION

The U.S. program for the disposal of civilian high-level radioactive waste (HLW) is governed by the Nuclear Waste Policy Act of 1982[1] and by the Nuclear Waste Policy Amendments Act of 1987[2].

These two laws describe the legal responsibilities of the various parties involved in the deep geological disposal of HLW.

The U.S. Department of Energy (DOE) has the responsibility for site characterization, repository design. construction, and operations, including waste emplacement, and eventual repository closing. Prior to initiation of repository construction, the DOE must submit to the U.S. Nuclear Regulatory Commission (NRC) an application for a license to receive and emplace HLW. The NRC may issue a repository construction authorization upon determining that the license application (LA) provides the necessary public health and safety safeguards. The rules governing repository licensing have been prescribed by the NRC in Part 60 of the U.S. Code of Federal Regulations.[3]

The site selected for site characterization is the Yucca Mountain site, in southern Nevada. about 160 kilometers northwest of the city of Las Vegas. Nevada (Fig. 1). Conceptual[4–5–6] designs for a repository at this site call for waste emplacement at a depth of slightly over 300 m. Access to the waste emplacement horizon will be provided by means of four vertical shafts and by means of two inclined ramps (Fig. 2). Two of the shafts, the exploratory shafts, are to be constructed during the early phase of site characterization. They will be utilized for geomechanical, hydrological, and geochemical characterization of the rock mass. as well as to provide access to a proposed in-situ test facility at the repository horizon. Regulatory concerns about the construction of the exploratory shafts arise from the fact that these shafts are designed to become part of the repository, and as such are subject to NRC licensing. In particular. they must comply with the regulatory requirement that "The program of site characterization shall be conducted… in such a manner as to limit adverse effects on the long-term performance of the geologic repository" (10 CFR Part 60.15 (d) (1)).[3]

The Department of Energy (DOE) is required by the Nuclear Waste Policy Act of 1982[1] and by 10 CFR Part 60[3] to conduct a program of site characterization before submitting a license application. The Exploratory Shaft Facility (ESF) will provide the access and support for exploration, in-situ testing and monitoring activities at the candidate horizon for the potential repository.

Before proceeding to sink shafts at any site. the DOE must submit. for review and comments, a site-characterization plan (SCP) to the Nuclear Regulatory Commission (NRC), the affected state. Indian Tribes and the public. This plan will describe the investigations and tests identified as necessary to adequately characterize the site. The NRC describes site characterization in 10 CFR 60.2[3] as:

> "…the program of exploration and research. both in the laboratory and in the field, undertaken to establish the geologic conditions and the ranges of those parameters of a particular site relevant to the procedures under this part. Site characterization includes borings, surface excavations, excavation of exploratory shafts, limited subsurface lateral excavations and borings, and in-situ testing at depth needed to determine the suitability of the site for a geologic repository…."

The geologic conditions at the candidate site will be investigated by means of surface-based investigations (e.g. seismic surveys), deep and shallow boreholes. laboratory tests, and. most important, in-situ testing and monitoring conducted in the host rock at the proposed depth of the repository. Based on the current design concept as detailed in the Consultation Draft Site Characterization Plan (CDSCP),[7] the ESF consists of surface support facilities, two shafts, and subsurface excavations (such as drifts. test rooms and borings). Laboratory data and surface findings will be integrated with in-situ test data to assess site suitability and verify parameters for repository design.

The in-situ testing for site characterization will be conducted in the shafts. in the main test area at repository depth. and in the exploratory drifts. It will start during shaft construction and will continue until sufficient data have been collected for a license application. These tests will concentrate on characterizing the rock mass. They should be designed to assess such characteristics as in-situ stress. discontinuities, thermomechanical parameters, geochemical properties, and hydrological properties.

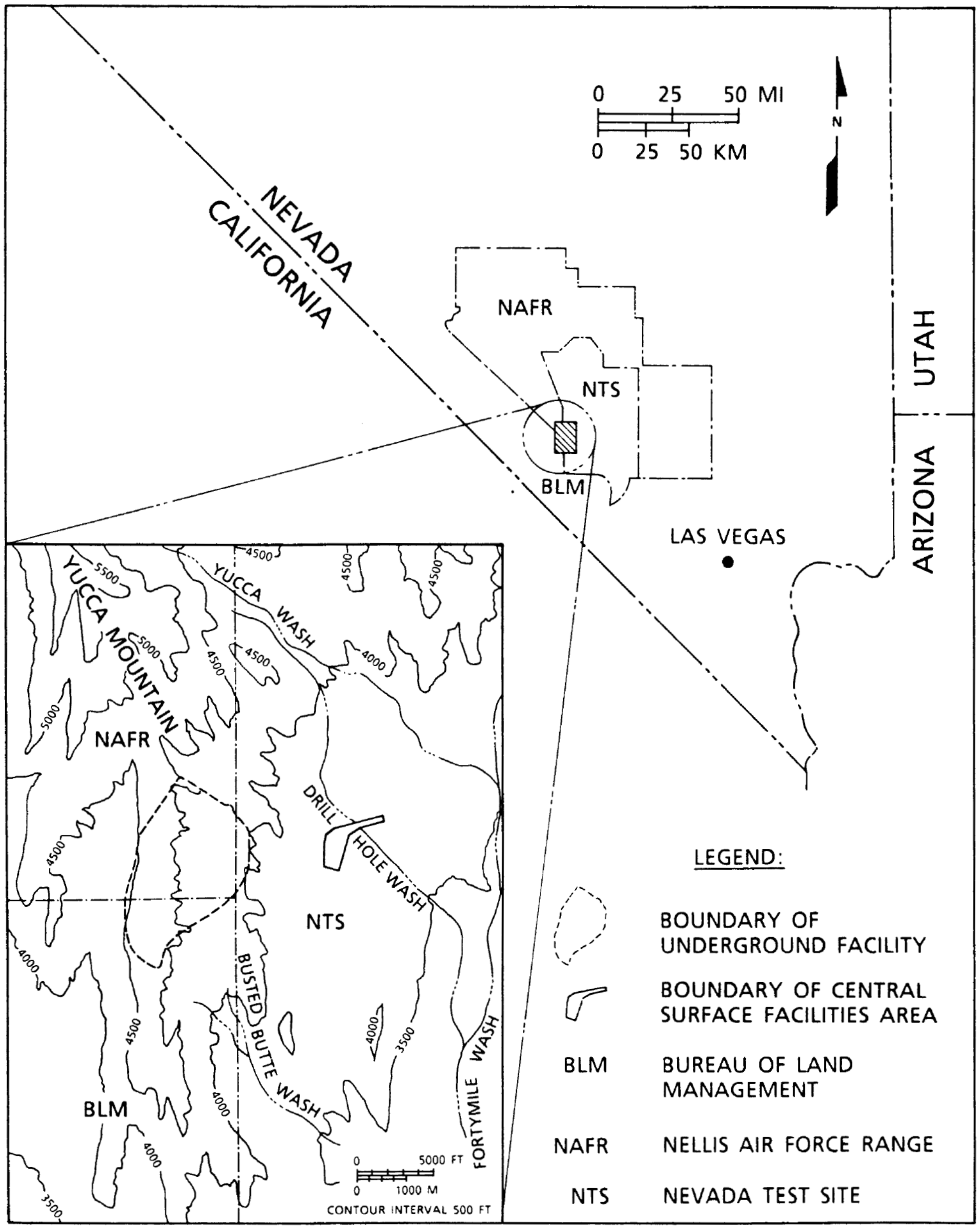

Fig. 1. Yucca Mountain, selected for site characterization for a possible high level waste repository. (From Site Characterization Plan Conceptual Design Report[5])

Since the exploratory shafts will become part of the future repository. they are subject to the same regulatory design requirements as the repository shafts. The NRC has two broad criteria for reviewing the DOE designs of the Exploratory Shaft Facility:

1. The ESF design and construction should limit adverse impact on long-term performance of the repository, i.e. should not compromise the waste isolation capacity of the geological barrier at the site.
2. The ESF design and construction should not preclude gathering adequate site characterization data. Because long-term prediction of the behavior of the geological barrier surrounding the repository will depend heavily on data gathered in the ESF, it is deemed particularly important that truly representative in-situ data be acquired, i.e. data that is not tainted by construction or interference effects.

This paper discusses NRC information needs with regard to design and construction considerations for the Exploratory Shaft Facility. NRC staff technical positions on Design Information Needs in the SCP,[8] on In 8 Situ Testing,[9] and on Borehole and Shaft Sealing[10] provide additional guidance in those specific areas.

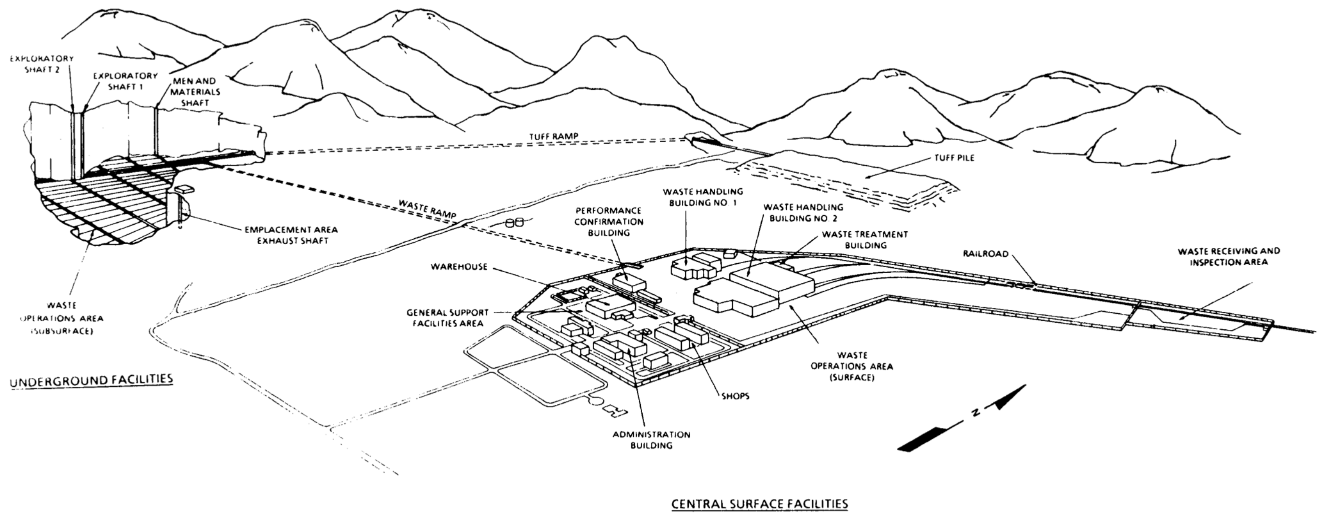

Fig. 2. Conceptual configuration of a HLW repository at the Yucca Mountain site. (From Site Characterization Plan Conceptual Design Report[5])

2.0
REGULATORY-FRAMEWORK

The NRC regulatory framework for the review of DOE's design and construction of the ESF consists of the following parts: 1) design and construction considerations, 2) site characterization considerations, 3) repository performance considerations, and 4) quality assurance considerations. The applicable 10 CFR Part 60[3] regulations are listed below.

2.1
Applicable Regulations: Design and Construction

Sections 60.131. 60.133 and 60.134 of 10 CFR Part 60[3] address the design criteria for the geologic repository operations area. Since the ESF may become a part of the repository. relevant portions of these sections are also applicable to the ESF design. §60.131 identifies the general design criteria for the geologic repository operations area. These include (a) radiological protection, and (b) structures. systems, and components important to safety. §60.133 lists additional design criteria for the underground facility, including general criteria for the underground facility. and criteria for flexibility of design. retrieval of waste. control of water and gas, underground openings. rock excavation, underground facility ventilation. engineered barriers, and thermal loads. §60.134 states the criteria for the design of seals for shafts and boreholes. This includes (a) a general design criterion. and (b) selection of materials and placement methods.

2.2
Applicable Regulations: Site Characterization

Section 60.15 of 10 CFR Part 60[3] addresses the site characterization requirements. In addition, Sections 60.140. 60.141. and 60.142 of Subpart F address the performance confirmation program requirements. The performance confirmation testing will be initiated during site characterization. Presumably, much of that testing will be conducted within the ESF. Specific technical issues to be addressed during this test phase include confirming geotechnical and design parameters, as well as design testing.

2.3
Applicable Regulations: Repository Performance

Section 60.112 of 10 CFR Part 60[3] addresses the requirements for selection of the geologic setting and design of the engineered barrier system and the shafts. boreholes and their seals to meet the overall system performance objective for the geologic repository after permanent closure. Both anticipated processes and events and unanticipated processes and events must be considered. A preliminary performance assessment will be required to assess how the ESF component of the repository contributes to the overall performance in meeting the Environmental Protection Agency standards.

2.4
Applicable Regulations: Quality Assurance

Subpart G of 10 CFR 60[3] requires that the DOE establish a Quality Assurance Program which includes quality control. Section 60.151 addresses the applicability: "The quality assurance program applies to all systems. structures and components important to safety. to design and characterization of barriers important to waste isolation and to activities related thereto. These activities include: site characterization. facility and equipment construction. facility operation, performance confirmation. permanent closure, and decontamination and dismantling of surface facilities."

Applicable portions of Appendix B of 10 CFR Part 50[11] are incorporated by reference in Subpart G. For items important to safety, important to waste isolation, and related activities, Criterion III of Appendix B, 10 CFR 50[11] requires that the DOE establish measures to assure that applicable regulatory requirements and design bases are correctly translated into specifications, drawings. procedures. and instructions.

3.0
TECHNICAL CONSIDERATIONS IN THE NRC REVIEW OF ESF DESIGN

As mentioned earlier, the ESF will be integrated into the repository. The NRC review of the license application will not take place until after the construction of the ESF and until site characterization testing has been completed. It is important to assure that no unmitigable damage will be inflicted on the geologic setting as a result of the ESF construction and site characterization efforts. Therefore, the NRC will review the DOE designs of the ESF using the applicable sections of 10 CFR 60 regulations as bases. The following sections highlight certain technical considerations related to design. construction, site characterization. performance analyses and quality assurance.

3.1
Design Considerations

The exploratory shafts could form one of the more direct pathways between the repository and the accessible environment. and will be the earlier repository structures to be developed. It is necessary that a conservative design approach be implemented to preclude shafts from becoming preferential radionuclide migration pathways. The shaft location aspects affecting radionuclide migration that need to be considered are the proximity to the areas designated for future waste emplacement. the overall repository geometry and the emplacement area drainage. The potential effects of repository conditions (such as thermal effects, subsidence or uplift) on the shafts and on the shaft seals should be considered in shaft location selection and shaft design.

The ESF design process and the approval process for the design should consider 10 CFR 60[3] requirements that deal with site characterization and long-term isolation. There should be clear and systematic documentation of how each relevant 10 CFR 60[3] requirement is translated into design bases. specifications, drawings. procedures. and instructions. As a part of the process, the applicable 10 CFR Part 60 requirements dealing with ESF design and construction should be identified. In addition. a verification process should assure that 10 CFR Part 60 requirements are incorporated into the various design stages.

The ESF should be designed to provide adequate site characterization without compromising the waste isolation capacity of the geological barrier.

The ESF permanent structures, systems, and components important to safety and/or waste isolation that are planned to be incorporated into the repository should be subjected to the same criteria. standards. and quality assurance levels as required for the repository. The exploratory shafts and drifts are likely to be such permanent structures. Site specific issues such as seismic. tectonic and climatic issues should be included in the design considerations.

The extent of the ESF and the location of ESF shafts, exploratory drifts. boreholes and tests should adequately contribute to establishing the geologic conditions and the ranges of the parameters important to repository performance and to site characterization.

Shaft location aspects that need to be considered include potential for infiltration, flooding and erosion, proximity to geologic faults. and proximity to areas designated for underground testing or future waste emplacement. The surface locations of the shafts should be selected so as to adequately limit the potential long-term infiltration of surface water through the damaged zone around the shafts and through the shaft backfill (after decommissioning).

The surface location of the exploratory shafts can be important because the location may affect the long-term infiltration of surface water through the damaged zone around the shafts or through the shaft backfill and seals into the waste emplacement area. Reasonable yet conservative estimates of infiltration. flooding. sheet flow and other potential water intrusions should be made, taking into account climatic changes that could result in additional rainfall and surface erosion. Uncertainties will always exist in these estimates. A prudent means of arriving at reasonable locations of shafts is to consider these uncertainties and, whenever possible, locate the openings away from flow channels. to locations where the potential for future infiltration into the shaft's damaged zone or backfill is low.

The ESF design should be compatible and consistent with the repository design with respect to underground location and layout of permanent ESF structures. systems and components. Permanent ESF structures, systems and components should be designed and constructed for sufficient maintainable design life period to provide for the waste retrieval option.

The permanent ESF underground openings should be designed to remain stable during operation and perform in a satisfactory manner under repository conditions. Because the shafts and drifts will be relatively close to the emplaced waste. they may become subjected to the combined thermal. mechanical. hydrological, and chemical effects induced by the waste. The shaft lining should be designed to provide stability of the opening under repository conditions during preclosure period and to provide for liner removal and permanent shaft sealing where needed at decommissioning.

The postclosure seals for shafts, drifts and boreholes should be considered in the ESF design and their tentative locations should be selected. The sections of the ESF shafts and drifts designated for sealing should be designed to be compatible with future repository seals. Large uncertainties are associated with the long-term performance of seals for shafts, drifts and boreholes. A prudent design should consider minimizing the reliance on repository seals by locating and excavating the shafts conservatively to limit water inflow and air outflow.

Horizontal exploratory excavations within the repository area can provide vital information on the geologic conditions and on the ranges of the parameters important to waste isolation, as well as to the design and construction of the repository. The design and construction of the exploratory drifts should provide for placement of seals or other control measures at the known geologic structures and anomalies such as faults or excessively fractured zones.

The portions of the shafts important to drainage (the portions below repository level) should be designed to provide adequate, reliable long-term drainage of any water infiltrating the damaged zone of the shaft. Reasonable assurance should be provided that such drainage zones will not allow accelerated travel of radionuclide-carrying water from the emplacement area to the accessible environment, e.g. to the water table.

To minimize groundwater flux through the waste emplacement area. a reliable long-term drainage should be designed at key locations. such as repository shafts. The portions of the shafts important to drainage (the portions of the shafts below repository level) should be designed to provide long-term drainage of any water infiltrating the damaged zone of the shaft.

The ESF drifts in the vicinity of the ESF shafts should be sloped to drain into the shafts to prevent water or moisture from draining into waste emplacement areas. Drainage design simultaneously must preclude facilitated migration of radionuclide-contaminated water to the water table.

3.2
Construction Considerations

Due to the uncertainties associated with prediction of long-term performance of natural barriers. seals. and drainage under repository conditions. a conservative excavation method should be used to minimize the damaged zone around exploratory shafts and drifts. certainly in those sections where sealing rather than drainage is the design objective. If the drill and blast method is used for construction of the exploratory shafts and drifts. the requirements to limit adverse impact on long-term performance of the repository and on the in-situ testing in shafts for site characterization should be accommodated to the extent it is necessary for meeting the design criteria of 10 CFR Part 60.[3]

Smooth wall blasting with strictly controlled procedures is an acceptable excavation method to limit the damage to the rock surrounding the shaft walls. A test blast program for the drill and blast shaft excavations should be considered at the onset of each new geological formation encountered during shaft sinking. Vibrations from blasting should be monitored and recorded and should be used as important information for blasting control and for damaged zone investigations.

The use of water for blast-hole drilling and other construction activities should be limited so that the characterization of the surrounding rock mass is not compromised.

The ground control measures implemented during construction affect shaft and drift wall deformations and the development of a modified permeability zone near the shaft. Reinforcement and support systems could affect permanent closure of the shafts upon completion of waste emplacement operations, and could also affect long-term isolation capabilities of the site, depending upon the mode of deterioration of such components.

Sections of ESF shafts and drifts designated for sealing should be excavated and supported in such a manner as to be compatible with future seals.

3.3
Site Characterization Considerations

The underground ESF should be designed and constructed such as to limit adverse impacts on site characterization. Sufficient separation distances should be established between the ESF construction or operations and in-situ tests to prevent interference. Likewise, large-scale tests should be designed and located such that there is minimal interference between any two tests.

ESF exploration and testing should not adversely affect overall site integrity as required by 10 CFR 60.112.[3]

The construction and operations should be compatible with extensive site data gathering requirements. to allow for sufficient range of site characterization activities such as geological mapping, stereophotography, geotechnical, thermomechanical, hydrological and geochemical testing.

3.4
Performance Analysis Considerations

The design and performance requirements for shafts. boreholes and their seals are governed by the Performance

Objectives of 10 CFR 60.112.[3] These requirements state that the geologic setting shall be selected and the engineered barrier system and the shafts, boreholes and their seals shall be designed to assure that releases of radioactive materials to the accessible environment following permanent closure conform to such generally applicable environmental standards for radioactivity as may have been established by the Environmental Protection Agency with respect to both anticipated processes and events and unanticipated processes and events. Accordingly, the ESF design of shafts, drifts, alcoves. drainage. seals and their construction should meet the post-closure performance requirements.

The design of the Exploratory Shaft Facility has to be completed without the benefit of the subsurface in-situ testing and exploration. Inevitably. there will be uncertainties associated with prediction of long-term performance of natural barriers, damaged zone around shafts, and drainage below the repository level. These uncertainties should be accounted for in the overall system performance analysis.

Conservative methods should be used when analyzing the ESF impact on the overall performance of the repository.

Shaft design and construction requirements are governed by performance analysis, which must provide reasonable assurance that the shafts will not allow excessive radionuclide releases. The regulatory requirements result in the need for careful control of numerous aspects of the design and construction of the whole ESP. The potential impact on long-term waste isolation of any characterization efforts, such as holes drilled from the shafts, drifts and alcoves, will need to be evaluated in order to assure that site characterization does not have unacceptable detrimental effects on the site isolation capability.

3.5
Quality Assurance Considerations

A quality assurance program in compliance with 10 CFR Part 60[3] Subpart G should be implemented for ESF design and construction activities. The QA program should identify those items and activities of the ESF which are potentially important to safety or waste isolation in accordance with the NRC guidance in NUREG-1318.[12]

Certain principal QA measures for design should be established to assure that applicable regulatory requirements and design bases are correctly translated into specifications, drawings, procedures, and instructions (as described in Criterion III of Appendix B. 10 CFR Part 50[11]). These measures include: control of design interfaces. design verification. control of design changes. and use of appropriate standards.

4.0
RECAPITULATION

Repository shafts form a potential preferential water inflow path and radionuclide migration path. For that reason their impact on repository performance is of regulatory concern. The exploratory shafts in particular, together with the associated excavations for the exploratory shaft facility. are of concern because they are among the first repository components to be developed. Their impact on repository performance may be permanent.

This paper presents an overview of the regulatory framework that needs to be accounted for in repository design and construction. and which applies to the exploratory shaft facility as well. The broad governing criteria are identified, and their development and application to various specific technical aspects of the ESF design and construction are illustrated.

REFERENCES

1. Nuclear Waste Policy Act of 1982 (NWPA). Public Law 97–425. 96 Stat. 2201, 42 USC 10101, Washington, D.C., January 1983.
2. Nuclear Waste Policy Amendments Act of 1987 (NWPAA). Public Law. Washington. D.C., December 1987.
3. U.S. Nuclear Regulatory Commission (NRC), Disposal of High-Level Radioactive Wastes in Geologic Repositories. Code of Federal Regulations, Energy, Title 10, Part 60, U.S. Government Printing Office, Washington, D.C.
4. Voegele, M.D. and C.P. Gertz, The Siting and Licensing of a Repository. Tunneling Technology Newsletter, No. 63, pp. 1–10. National Research Council, U.S. National Committee on Tunneling Technology. Washington, D.C., September 1988.
5. MacDougall. H.R., L.W.Scully. and J.R.Tillerson, Compilers. Site Characterization Plan Conceptual Design Report. Sandia Report SAND 84–2641. Prepared for the U.S. Department of Energy by Sandia National Laboratories, Albuquerque. N.M., and Livermore. Ca.
6. U.S. Department of Energy. Environmental Assessment Overview— Yucca Mountain Site, Nevada Research and Development Area. Nevada. DOE/RW-0079. Office of Civilian Radioactive Waste Management, Washington. D.C., May 1986.
7. U.S. Department of Energy. Consultation Draft Site Characterization Plan, Yucca Mountain Site, Nevada Research and Development Area, Nevada. Office of Civilian Radioactive Waste Management, Washington, D.C., 1988.
8. NRC. Generic Technical Position on Design Information Needs in the Site Characterization Plan. U.S. Nuclear Regulatory Commission, Washington, D.C., 1985.
9. NRC. Generic Technical Position on In Situ Testing During Site Characterization for High-Level Nuclear Waste Repositories. U.S. Nuclear Regulatory Commission, Washington, D.C., 1985.
10. NRC. Generic Technical Position on Borehole and Shaft Sealing of High-Level Nuclear Waste Repositories. Compiled by Engineering Branch. Division of Waste Management. U.S. Nuclear Regulatory Commission. Washington. D.C., 1986.
11. U.S. Nuclear Regulatory Commission. Domestic Licensing of Production and Utilization Facilities. Code of Federal Regulations, Energy. Title 10, Part 50. U.S. Government Printing Office, Washington, D.C.
12. Duncan, A.B., S.G.Bilhorn, and J.E.Kennedy. Technical Position on Items and Activities in the High-level Waste Geologic Repository Program Subject to Quality Assurance Requirements. NUREG-1318. U.S. Nuclear Regulatory Commission, Washington. D.C., 1988.

State of the art in blind shaft drilling

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SYNOPSYS

The traditional methods of blind shaft drilling are described together with the limitations that these impose on cutter loading, bottom hole cleaning and shaft verticality.

The construction and use of pilot holes for the guidance of full face blind shaft drill bits are outlined, as are the advantages to be derived from their use.

Recent advances in the design of steerable bottom hole assemblies are on the verge of allowing production of a new generation of blind shaft drilling machines which will enable larger, straighter shafts to be drilled more quickly and cheaper.

The merits of various shapes of bit, including flat, hemi-spherical and conical are reviewed with clear conclusions being drawn for bits working with reverse circulation in sedimentary rocks.

The use of polymer muds for drilling shale formations and clay now enables blind shaft drilling to be more competatively carried out in these types of strata.

An analysis of future prospects for blind shaft drilling is made with the confident expectation that during the next two decades this method of shaft construction will progressively increase its share of the market for shafts up to 26 feet in diameter.

TRADITIONAL METHODS

Blind shaft drilling using forward and reverse circulation mud flush has been carried out by mechanical rotary rigs for more than 100 years. During this time the drilling tools and rigs have become larger, more sophisticated and reliable but the methods of drilling have remained virtually unchanged.

Basically a surface mounted rig turns drill pipe which is connected to the bit. The bit is turned while extra drill collars or weights stacked on the drill pipe just above the bit are employed to provide extra downward thrust. The drill string, consisting of bit, weights, stabilisers and drill pipe, weighs far more than the load applied to the bit and the extra weight is held back so as to keep the drill pipe in tension and to provide a “pendulum” righting force that is used to assist in preventing the hole from deviating from the vertical.

This system has been used to blind drill many holes in all parts of the world including very large shafts of over 20 foot in diameter in Russia and China and some very deep shafts including the 6250 foot deep hole on Amchitka Island, Alaska. The most common application however has been for shafts of less than 10 foot in diameter to depths of less than 1000 feet.

Blind shaft drilling employs bits dressed with cutters of various types including toothed, carbide insert and disc cutters. The rock at the base of the shaft is broken by the cutter gouging, scraping or crushing the rock. The rock chips or cuttings are removed by the flow of the circulation fluid passing over the base. In general the above process has a very low efficiency factor because inadequate load is applied to the cutters to cause deep penetration of the rock by the cutting structure. Shallow penetration results in the production of small rock chips or cuttings with the consequential increase in energy requirements as it takes much more energy to break rock into small pieces than into large ones.

In addition the cleaning of the cuttings from the bottom of the hole has generally been quite poor with cutters passing over previously cut rock thereby causing re-grinding and further degradation of the rock. The loose cuttings on the bottom of the hole provide a cushion to the cutters which means that not all of the available load is applied to cut new rock but some is utilised in regrinding the old cuttings. The energy required to break rock into small pieces using rolling cutters is very high and this with the requirement to hold back a significant portion of the total drill string weight has imposed a practical and economic limit on the size of shaft that may be drilled by the blind shaft method.

There is a continuing demand for relatively small shafts of less than 10 foot diameter and as these are quite difficult to construct using conventional mining methods it is natural that they should be formed by blind drilling techniques. In order for blind shaft drilling to be more widely used it will be necessary for the system to be able to form larger shafts at economic rates.

At present most shaft drilling rigs operate with a significant proportion of the drill string weight held back so as to keep the drill pipe in tension and to provide a “pendulum” righting force used to keep the hole vertical. The righting force is never very large in relation to the downward thrust and torque employed but for deep holes this corrective force becomes relatively small and of little significance. For example consider a shaft 1000 feet deep with the bit 1 foot off vertical, the corrective force is only l/1000th of the hold back force. Even on a large rig the hold back force is unlikely to much exceed 500,000 pounds so the righting force is only 500 pounds. The cost of providing the extra weight, stronger drill pipe, heavier rig and more elaborate site preparation is very high indeed for the marginal benefit gained and as shafts become larger in diameter and deeper in depth the effects of the righting force become even less significant. Even with much shallower and slimmer shafts deviation of the drilled hole from the vertical is very common and is difficult and very expensive to correct. The requirement of clients for shafts with guaranteed verticality frequently bars the use of blind shaft drilling as the method of construction.

PILOT HOLE DRILLING

Significant advances have recently been made in the use of downhole instrumentation for directional control or steering of small diameter boreholes. If these boreholes can be drilled straight, vertical and of suitable size, say 12 to 20 inches in diameter then they may be used as pilot guide holes for a large diameter blind hole drill bit fitted with a guide “stinger”.

The use of such pilot holes offers the advantage that very little drill string weight needs to be held back as no “pendulum” force is required to guide the bit. The bit is guided by the “stinger” which fits into, and follows, the pilot hole. The extra available weight may then be used to apply larger loadings to the cutters on the bit thereby achieving deeper penetration of the cutters into the rock, the generation of larger cuttings, an increase in penetration rate and a reduction in cutter cost per unit volume of rock excavated.

The technique of using a forward circulation conventionally drilled pilot hole as a guide for a larger bit has been used very successfully by competitive contractors for much of the 1980s. The knowledge of how to design and build suitable downhole tools, bits and hole openers for use with pilot holes is well established for shafts upto 12 or 14 foot in diameter. The combination of ultra straight pilot holes formed with down-the-hole steering equipment and hole opening with large capacity shaft boring rigs should therefore enable shafts with diameters up to 20 feet to be competitively drilled in many sedimentary rocks.

STEERABLE ASSEMBLIES

Some of the latest developments in blind shaft drilling involve the use of steerable bottom hole drilling assemblies. The position and attitude of the bottom hole assembly is measured by down-the-hole instruments. The attitude of the bottom hole assembly may then be adjusted by movable rams on the assembly so as to steer the bit and assembly in the required direction. By the constant monitoring and frequent adjustments to these rams the hole can be drilled very straight and kept near to the vertical, even in variable or deeply dipping strata.

At the time of writing design work is known to have taken place in the United States of America, mainland Europe and Great Britain on various versions of full face, steerable, shaft drilling equipment. In Great Britain in 1985 trials were conducted using a 12.5 foot diameter steerable assembly operated from a conventional rotary table rig. Further work is progressing on a full face steerable machine with down thrust being obtained from friction pads that grip the shaft wall. In Germany a steerable machine with downhole motors to turn the bit has been built and trials with it have recently been undertaken both in Germany and in Russia. In the U.S.A. a machine is currently in the course of construction and it is anticipated that it will see service during the early part of 1989 in Australia.

The objective of using these new generation machines is to drill straighter, more vertical holes and to enable higher loadings to be applied to the cutters on the bit. Steady developments in the design of cutters, particularly disc cutters, for use with tunnelling machines now enable design loads of more than 30 tonnes to be applied to individual cutters. Previously these loadings have rarely, if ever, been achieved in shaft drilling applications when using the conventional holdback technique.

If the direction of drilling can be controlled without the need for holdback then for any particular rig hook load capacity the available weight that may be used for cutting and the size of hole that can be drilled is increased or alternatively a faster rate of drilling should be achievable.

An increase in cutter loadings results in an increase in torque and there is an obvious attraction in applying the rotary motion to the bit by way of downhole motors reacting either through a static drill pipe or reacting against the shaft wall itself. The need to do this becomes even greater when the diameter of the drilled shaft increases from the typical 12 or 14 foot maximum that is economically achievable today to the anticipated 20 to 26 foot diameter that is likely to be possible using these techniques.

BIT SHAPE

Much discussion still takes place concerning the profile of the bit. Comparative experience only exists for bits operated with the conventional holdback system of drilling. Some drillers claim that a flat profile bit is easier to steer and that it will drill a straighter hole than will a bit of hemispherical or conical shape. At the present time there is not enough definitive evidence existing to determine which shape of bit is best for drilling a straight, vertical hole in any particular set of ground conditions.

The author does however believe that there are significant advantages to be gained in cuttings removal and bottom hole cleaning from the use of conical or hemispherical (shaped) bits when reverse circulation is employed. The reason for this is that the flow of the circulating fluid is from the perimeter of the bit towards the bit centre and if the profile of the hole bottom, as cut by the bit, is hemispherical or conical then the cuttings can move downhill while travelling towards the fluid pick up point. It is much easier for a cutting to travel downhill than for it to travel horizontally and consequently for any particular flowrate larger cuttings may be transported across the sloping base of a conical hole than may be transported across a flat or horizontal surface.

The majority of shafts are formed through strata that is bedded horizontally or nearly so. If there are hard bands within the main body of the rock then it has been found to be advantageous to use shaped bits for they enable the central part of the bit to penetrate the hard bed first. Different parts of the bit encounter the hard bed as the hole is deepened. For any particular thrust applied to the bit this process enables a higher loading to be applied to those cutters in contact with the hard bed than would be the case if all cutters contacted the hard bed simultaneously, as would occur with a flat bit encountering a hard horizontal or near horizontal bed. This enables a shaped bit to penetrate harder bedded strata than could a flat bit subjected to the same downward thrust.

DRILLING MUD

Great advances have been made in mud technology during the last decade and these new muds are progressively becoming more available for use in shaft drilling. It is not necessarily the hardest rock that is the most difficult to drill. Often it is the clays, seatearths and reactive shales that present the greatest challenge to blind shaft drilling. Several specialist polymers have been developed to assist in the drilling of these formations for they act to encapsulate the cuttings and cutters thereby prohibiting or at least delaying the swelling of the clays and the balling of the bit.

These polymers are used to make “low solids” muds which have very low densities. These muds exert less hydrostatic pressure upon the base of the hole than conventional bentonite based muds and in consequence the rock chips are more free to separate from the parent rock thereby leading to better hole cleaning and a faster rate of penetration. It is therefore often of advantage to use these polymer muds in preference to water or bentonite muds except where the ground conditions demand the formation of a physical filter cake to prevent the loss of mud into the strata.

UTILISATION

Studies of rig use have shown that it is not un-typical for a shaft drilling rig to drill for only about half the time it is on site even though the site operates on a 24 hours per day basis. The remainder of the period is occupied with site preparation and rigging up, weekend and holiday breaks, hole surveying, casing running, cementing and demobilisation as well as some time for maintenance or repair.

Major increases in overall productivity could be achieved if down-the-hole steerable assemblies could be perfected for then smaller rigs could be employed to drill larger holes and generally less site preparation is required for a small rig than for a large one. The latest medium capacity rigs are highly mobile and require very little time to erect and commission. This is a significant advantage over the massive, heavy duty rigs that have traditionally been used for the holdback drilling system. The application of larger loads to the cutters could dramatically increase the penetration rate in medium and hard strata and the creation of larger cuttings assists with maintenance and cleaning of the mud. In addition, with the steerable assembly the surveying of the hole is continuously carried out as the hole is being drilled and the drilling does not need to be interrupted to carry out a separate survey of the shape of the hole.

FUTURE PROSPECTS

It therefore seems likely that the worldwide future for shaft drilling is brighter than it has been for many years for the ability to drill straighter, more vertical shafts of larger diameter will undoubtably increase the appeal of this system to clients with the consequential increase in orders.

With the traditional shaft drilling techniques that have been available for some time shaft drilling has, generally, only been competitive for the construction of small or medium (less than 10 foot) diameter shafts or shafts where conventional mining techniques had to be supplemented by special procedures such as ground freezing or where the time for construction was particularly short. It will not be possible to make large improvements in the conventional mining techniques used for the construction of small shafts simply because of the very restricted space available for work at the base of a small diameter shaft. It is also thought unlikely that mining techniques can be much improved for the sinking of shafts up to say 26 feet in diameter because there is already a very great experience of this type of work and the tasks of forming shotholes and mucking out are already almost totally mechanised.

Blind shaft drilling will continue to be the predominant method used for the construction of small diameter blind holes and as the equipment and techniques for maintaining shaft verticality improve it is forecast that it will be used increasingly for the construction of larger and deeper holes of say upto 26 foot diameter, in direct competition with conventional shaft sinking.

Shaft sealing for nuclear waste repositories

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SYNOPSIS

Long-live radioactive waste can safely be isolated from human environment in underground repositories located in deep geological formations. In the multi-barrier concept shaft sealings play an important role to avert the migration of nuclides from the repositories to the biosphere. New sealing systems developed in the last decade are characterized by the following:

- distinction between the static abutment and the sealing element.
- calculation of the static abutments by numerical program systems.
- slippery bearing of the static abutment.
- application of clay and clay-sand mixture as well as asphalt and bitumen sealings.

With regard to repository safety the calculation and design of the sealings have to fit the requirements as follows.

- impermeability against liquids.
- corrosion resistance over very long periods (thousands of years).
- consideration of extra loading cases.
- minimum failure probability of the entire construction.

PURPOSE OF SHAFT SEALINGS FOR NUCLEAR WASTE REPOSITORIES

The disposal of nuclear waste in appropriate deep-seated strata is a suitable method for safe disposal of nuclear waste over long periods (medium radioactive waste 10^3 years, highly radioactive waste 10^5 years) because of very slow migration of materials in the lithosphere, ie to exclude the transport of dangerous amounts of radioactive nuclides to the biosphere or, at least, to restrict it to permissible limits. Within the multi-barrier concept the hermetic sealing of artificial connections between repository and surface level (shafts, if necessary also boreholes) after disposal is of decisive importance. The demands on the sealings can be summarized as follows:

- to avert inflow through the shaft into the repository.
- to avert migration from the repository through the shaft to upper strata, the aquifer and the biosphere.
- to avert migration through the possibly permeable excavation-affected rock area.
- if necessary insertion of additional sorptional elements to reduce the transport of radioactive nuclides in case of permeability.

These demands have to be met durably and maintenance-free. The periods to consider range between 10^3 and 108 years. Because of the importance and the extremely long operational periods the following premises have to be regarded in design of the sealings.

- impermeability against liquids.
- extremely high safety level (minimum failure probability).
- use of tried and tested constructional elements.
- time-dependent material deteriorations have to be excluded, changing conditions over years should, if possible, favour the loading condition and sealing effect.
- after a certain period of supervision it must be possible to leave the system without human interference.

These findings can also be applied—in various forms—for horizontal sealings and sealing systems for other purposes (eg. underground inflow protection, gas and compressed air storages, waste repositories, shaft plugging) [1, 2, 3, 4, 5].

PRINCIPLES OF ACTION AND RECOMMENDATION FOR DESIGN

The sealing constructions are divided into the static abutment for the transfer of forces to the rock and the sealing system to secure the impermeability against liquids. But, because of the extremely long operational periods, the sealing system has to guarantee durable corrosion resistance of the static concrete abutment.

Static abutment [1]

Since a complete stress determination by means of analytic methods is impossible, calculation of the abutments was performed by two and three dimensional FE programs based on the real behaviour of abutment material and rock (material laws, material parameters). Some of these material laws are presented in Fig 1. The application of inelastic material laws adapted to concrete also permits a more realistic estimation of bearing capacity and safety level not only for static abutments, but also for other concrete constructions (eg shaft linings), and thus, a more economical design. Using the inelastic method of design, these concrete shaft linings with large internal diameters (≥6.0 m) can even be used under uniformly distributed pressure around circumference (hydrostatic water pressure) of ≥ (5 to 6) MPa [6]. The analysis of the comprehensive FE calculations of abutment and rock loading shows that in previously applied rock-connected abutments the tensile stress was very high, independent of their geometry, and resulted in roof breaks and cracks in the concrete under axial fluid pressure of > (2 to 4) MPa. The tensile stresses reach their maximum at the contact point with the rock at the pressure side, and can not essentially be reduced by lengthening of the abutment (Fig 2). By means of systematic arrangement of slip planes in the contact area of abutment and rock (mobile contact, only with compressive stress, without shear stress) and by means of a special geometrical abutment shape the stress can be directed and favourably influenced so that only compressive stress occurs in the abutment (Fig 3). For slip plane materials plastic foils, epoxy coatings (epoxy resin, polyester, polyurethane), sprayed bituminous latex, mastic asphalt plates and sealing webs coated with bitumen are suitable. Materials based on bitumen and asphalt are in particular suited for this purpose since they have an additional sealing effect, and are marked by longevity and corrosion resistance. Abutment geometry, abutment length, necessary concrete quality and possibly necessary load-bearing reinforcement depend upon a number of parameters, such as fluid pressure, time-dependent rock pressure development, virgin stress state of the rock, excavation cross section, stress strain behavior of rock and abutment material. These parameters can be determined for definite cases only. Fig 4 presents some representative slippery abutment geometries, such as spherical calottes, truncated cone shaped and toothed abutments, Fig 5 shows an abutment construction for axial stress independent of direction which is especially suitable for application in rock strata with time-dependent high rock pressure (eg deep-seated salinar rocks). Here, a step-like geometry leads to a favourable stress state in the whole abutment caused by the rock pressure. Thus, possible axial tensile stresses caused by notch or other effects are negated [7].

On the basis of some hundreds of FE calculations regulations have been formulated which serve the predimensioning. The stability has finally to be proved by FE calculations. These checking calculations are performed for usual basic loading cases (BLC) (according to the conditions these are fluid pressure only from above, only from below, from below and above, without or with possibly time-dependent radial rock pressure). In the case of sealing of nuclear waste repositories anticipation of possible unfavourable conditions is indispensible because of the extreme longevity and the necessary high safety level. Therefore, stress caused by extra loading cases (ELC) has to be added to that of the basic loading cases. These extra loading cases are very unlikely to occur. They occur only for a short time if ever, or there is no verified data to be taken into account. Thus, they have so far been taken into account by means of the general safety coefficient. The following extra loading cases are of particular importance. In general, the slip plane has a favourable influence on the stress state under such extra loading cases.

ELC 1–
Thermal stress caused by cement hydration heat [8]

In massive concrete constructions the heat released in the hardening process can not be eliminated as soon as necessary, thus, the temperature in the core of the construction rises considerably. The irregular temporally changing temperature field leads to irregular thermal expansion of the concrete which results, in connection with the hardening process (change of Young's modulus), in considerable temporally changing internal stress. By means of a three-dimensional simulation program the temperature distribution was determined, and afterwards the internal stress was calculated by numerical integration. First, the internal stress in the core was tangential compressive stress which turned into tangential tensile stress after a few months, in the margin area it was contrary (Fig 6). Because of the locally dependent temporally changing thermal stress load-bearing

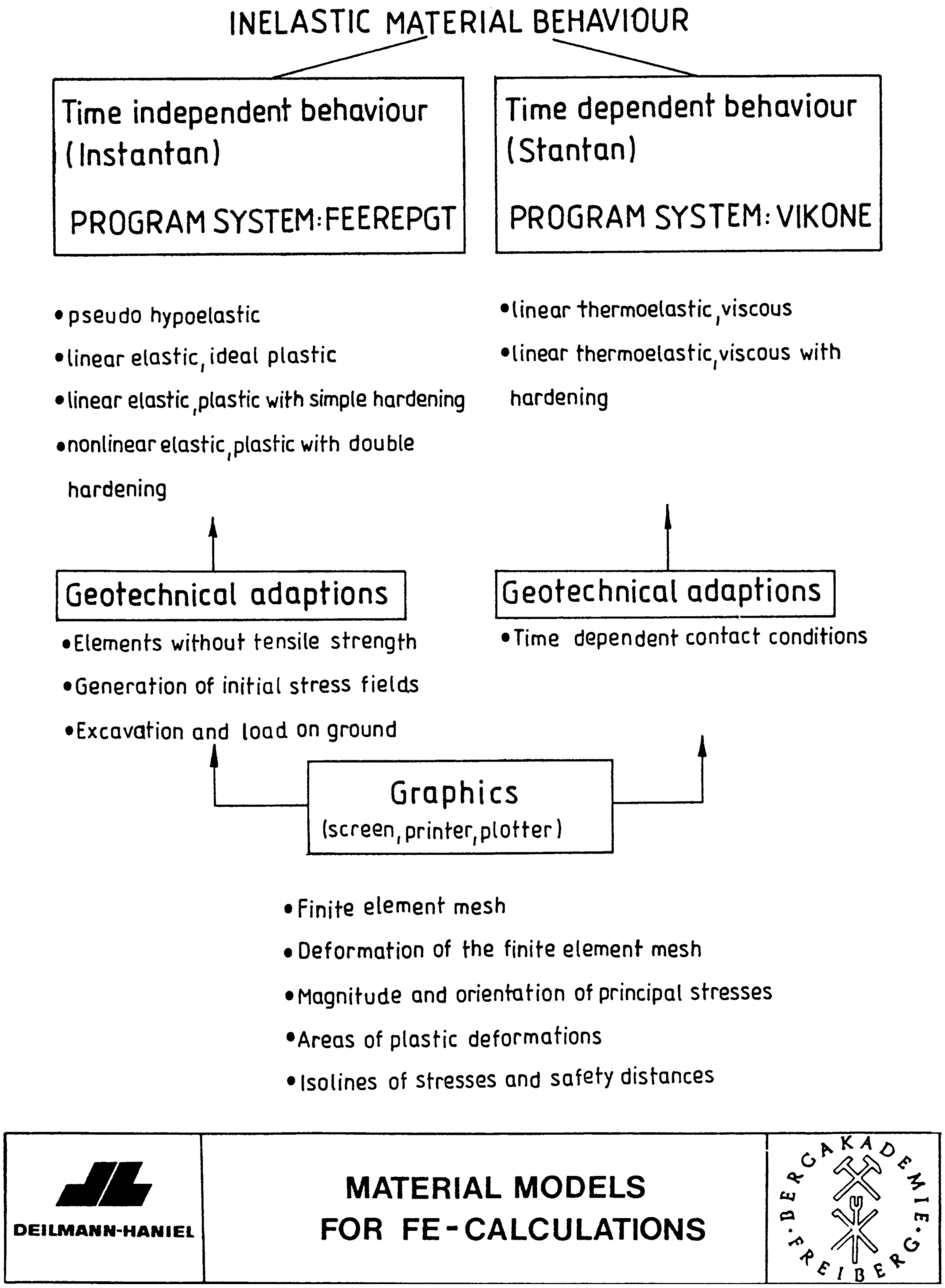

Fig 1

steel reinforcement might be necessary. Furthermore, the calculations show that the slip plane favourably influences the stress condition since it eliminates stress caused by hindered deformation and has a heat-insulating effect. A positive effect is also created by thermal insulation of the faces, low Young's modulus, low coefficient of thermal expansion as well as by purposeful addition of hardening accelorators and retarders [9].

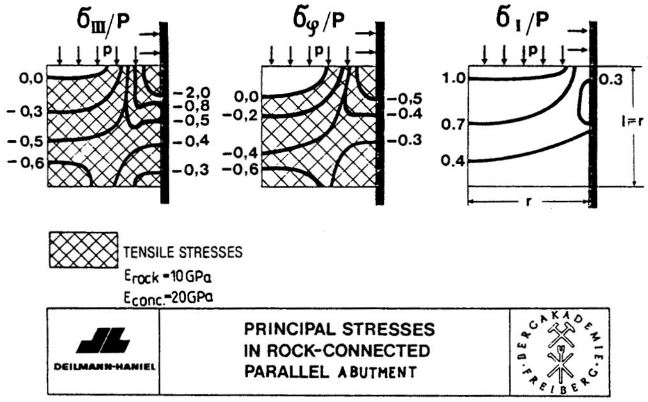

Fig 2

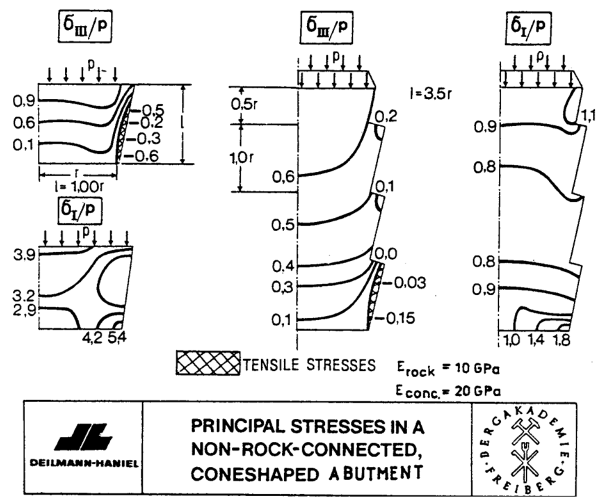

Fig 3

ELC 2–
Thermal stress caused by hot asphalt emplaced on the abutment

For sealing and corrosion resistance hots asphalt of (100 to 140)°C and great thickness is emplaced on the abutment. A (3 to 5) m thick cold layer of mastic asphalt or sand asphalt is arranged on the abutment and reduces the temperature changes in the abutment to a maximum of (2 to 3)°C. Thus, thermal stress in the abutment caused by the hot asphalt is almost completely eliminated and only thermal stress (additional tangential compressive stress) in the rock has to be considered, which is, however, decreased with cooling of the asphalt (within a few years).

ELC 3–
Stress caused by rock movement

Horizontal and vertical deformation fields formed around the abutment due to the convergence of the repository excavations in dependence on time have to be diagnosed and converted into additional rock stress fields. From the latter the stress state formed in the abutment can be calculated (Fig 7). In this connection, the stress-reducing effect of the slip plane (no transmission of shear and tensile stress in the contact area) has to be considered. There are unfavourable conditions if deformations with different signs (elongation, compression) occur in direction of the principal stress axes. The use of abutment materials with a low Young’s modulus has a positive effect, too.

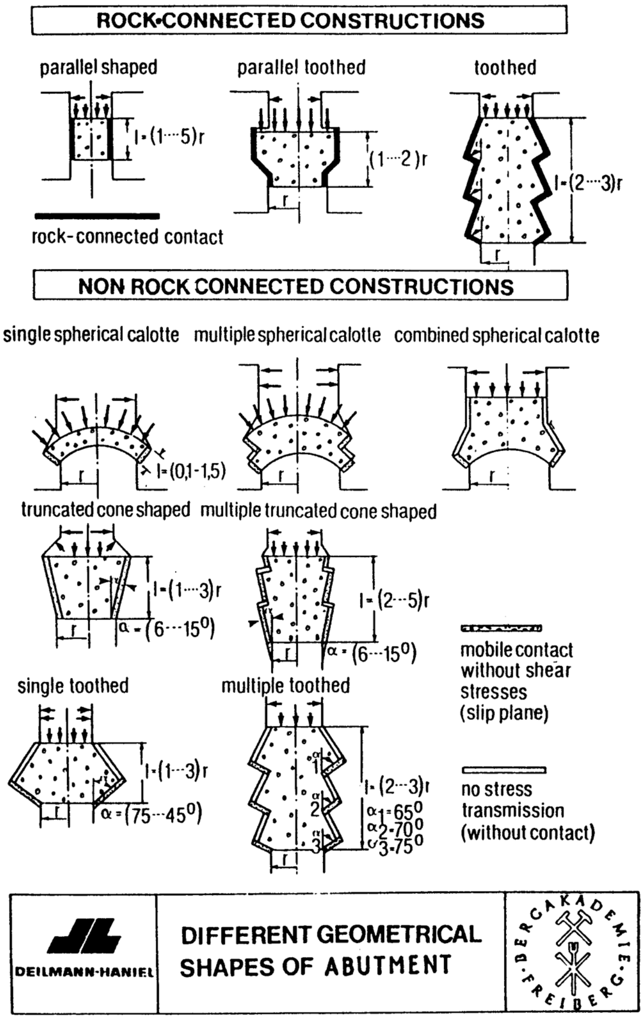

Fig 4

ELC 4–
Stress caused by earthquakes

There are no regulations or generally excepted calculation procedures, standards etc. for seismic loading and the interaction between excavation and underground construction. The evaluation of earthquake effects on underground excavations and constructions showed that even strong earthquakes had no or only few damaging effect in contrast to that on surface constructions [10,11]. The calculation of additional stress in the abutment and adjacent rock caused by earthquakes is possible by means of dynamic models if the respective parameters are available (eg maximum vibration velocity and maximum acceleration of the earthquake, dynamic Young's modulus and seismic wave velocity of the different geological strata and the sealing material). Under normal conditions the calculations can be restricted to vertically propagating compression and shear waves. Since the stress increases only for a very short time in this case, a higher stress state is permissible than in other loading cases. Calculation examples with dynamic programs verified findings of FE calculations performed, for example, in the USA [11] and Japan [12,13], ie additional dynamic stress caused by accelerations of up to 0.3 g has only very low influence on the stability.

For nuclear waste disposal some other extra loading cases have possibly to be considered with regard to abutment stress, eg evolution of heat in larger parts of the repository resulting in thermal stress caused by temperature gradients, deterioration of rock and constructional material parameters and mechanical stress due to thermal expansion in large rock areas. These cases, however, are not considered further.

Sealing systems and systems for corrosion protection

The sealing system which is also to protect the abutment concrete durably from corrosion has to seal large cross sections as well as the contact area of abutment and rock (this effect is supported by the contact pressure due to slip plane). It has also to

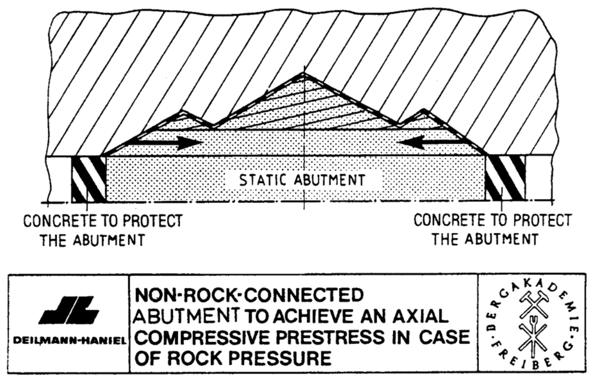

Fig 5

secure the impermeability of the excavation-affected rock area (Fig 8). This rock impermeability is necessary to avoid migration caused by axial flow in the excavation-affected, possibly loosened rock area. Impermeability against fluids from above and rock impermeability can be achieved by a purposeful arrangement of sealing packages of clay-additive mixtures, pure clay, sand asphalt, mastic asphalt as well as by asphalt packages of a thickness of at least 10 m. Clay packages alone can not fit the requirements of shaft sealing of nuclear waste repositiories (absolute impermeability over extremely long periods) since there are a number of negative effects caused eg by high temperatures (eg evolution of heat in the waste), mineralized waters (enormous increase of permeability, reduction or elimination of the swelling effect), and erosion. The addition of special modified bitumens and asphalts is an indispensable necessity.

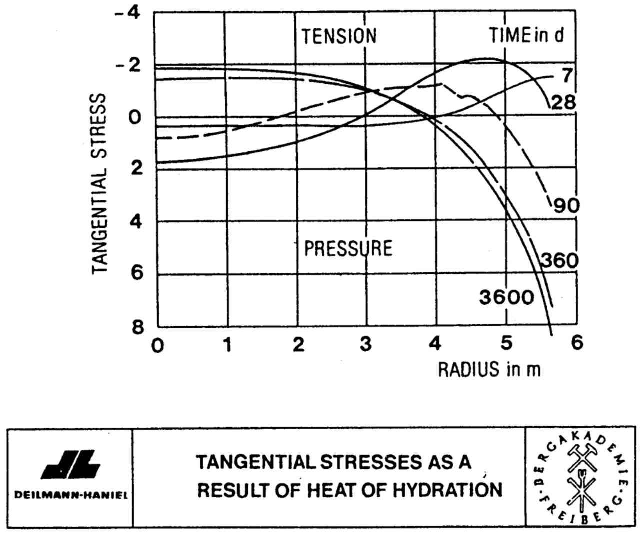

Fig 6

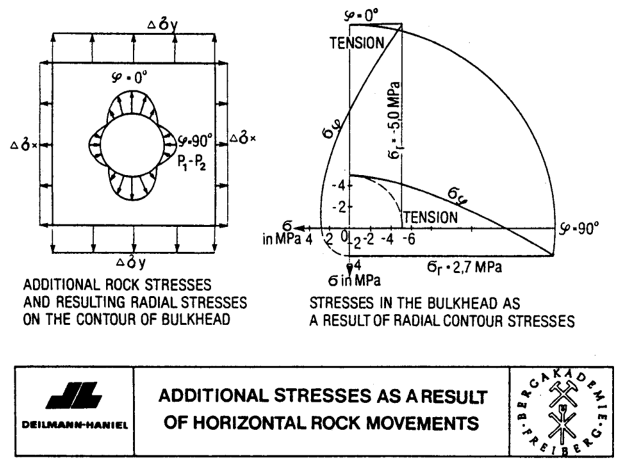

Fig 7

Mostly, it is necessary to completely fill the shaft up to surface level with asphalt, possibly by backfilling. The choice of the asphalt density has to secure a hydrostatic asphalt pressure which is always higher than that of the waters in the rock. Thus, the asphalt has an active sealing effect since it penetrates into joints, fractures, cracks, microcracks and pores in the rock. In addition to that, in flooded mine openings the asphalt pressure has to be higher than the fluid or gas pressure at the abutment from below. The area below the abutment has to be filled with inert material. Immediately under the abutment bitumen and asphalt layers have to be located the density of which facilitates their floating if the opening is flooded. This guarantees sealing effect and corrosion protection at the bottom of the abutment. Detailed design of the sealing system with regard to material selection (distillation bitumen, oxidized bitumens, modified bitumens, asphalts, clays), material quality (bitumens B 15 to B 200, kind, grain size and amounts of filling materials), arrangement and thickness of the single sealing packages is possible only if the geological, mineralogical, hydrological, geomechanical and geothermal conditions are well known.

At present, comprehensive theoretical considerations and laboratory investigations in terms of asphalt and bitumen are concentrated on the following problems.

1. Rheological parameters of different bitumens and asphalts since the description of these materials by the standard parameters (softening temperature., ductility, breaking point) is insufficient to assess their applicability in the sealing system.
2. Sedimentation of the fillers (single grain, particle swarm) in the asphalt in the course of the very slow cooling process and as long-term process in the cooled asphalt. Below a certain viscosity the sedimentation in the Newtonian fluid bitumen does not agree with the Stoke’s law.

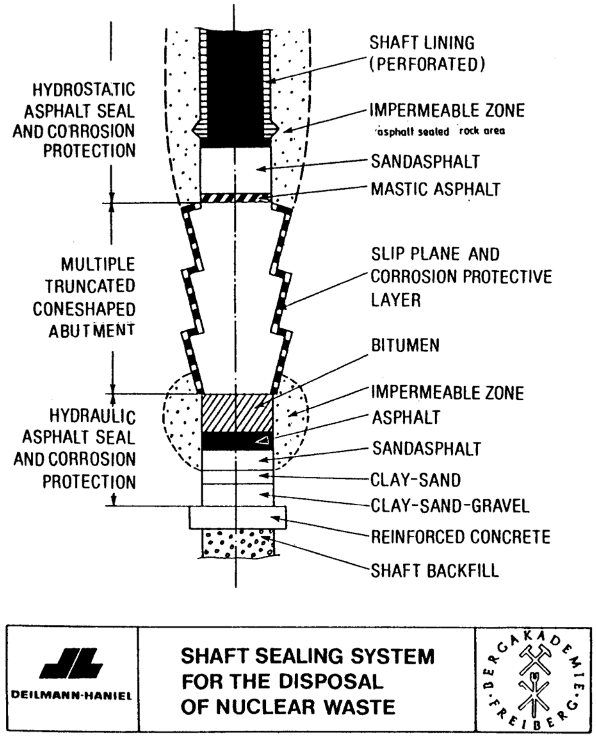

Fig 8

3. Temporary retardation of pressure propagation in long bitumen and asphalt columns with an elastic foundation. This temporary retardation is important, for example, to evaluate the effects of abutment displacement due to rock movement, earthquakes or other processes.
4. Time-dependent penetration of bitumen and asphalt into fracture and crack systems of different geometry (flow into a fracture) and plugging effects of fillers to assess impermeability effects and time-dependent bitumen and asphalt losses. Modification of the bitumens by means of fillers to obtain an initial yield point which excludes bitumen and asphalt losses in general.
5. Adhesion of bitumen and asphalt for different kinds of rock, measures to achieve good adhesion, in particular at highly hygroscopic salt surfaces (eg carnallitite) [14],
6. Diffusion of several gaseous media into the bitumen and alteration of the bitumen characteristics due to the influence of gaseous hydrocarbons (eg action of methane on the side of the sealing system facing the repository).

All these investigations explicitly lead to the conclusion that the selection of suitable, possibly modified bitumens and asphalts is the prerequisite to fit the requirements on sealing and corrosion protection of materials for sealing systems of nuclear waste repositories.

INVESTIGATIONS OF LONGEVITY, DAMAGE AND SAFETY OF THE ENTIRE SYSTEM

The applied materials have to fit the requirements of very low permeability i.e. liquid permeability, fracture-free adaption to rock and abutment movement and longevity (10^3 to 10^5 years). Because of uncertain longevity as well as unsolved technological problems metals, plastics, ceramic materials and molten salts can not be used. The application is restricted to clay, bitumen, asphalt and mineral additives or fillers. The ageing resistance of clay and clay-additive mixtures which have a sealing effect because of their low permeability, swelling capacity, plugging effect and plastic behavior has been verified for geological ages. However, the constructional design has to exclude erosional and high-temperature effects ($T \geq 90°C$). Bitumens and asphalts are durably resistant against all possible liquid and gaseous media. Their ideal suitability for extremely long periods is verified not only by theoretical considerations and laboratory investigations, but also by natural asphalt occurrences and buildings constructed about 5000 years ago at the rivers Euphrates and Tigris [15, 16]. Theoretically possible minor ageing effects can be neglected because of the great thickness of the asphalt packages. In this connection, stable thermal conditions, lacking exposure to sunlight and nearly complete exclusion of air have an additional positive effect. Concrete as abutment material is protected from aqueous solutions which are the main corrosive agents by thick bitumen and asphalt packages, slipping layers at the periphery and by the emplacement in tight fracture-free rock. Concrete buildings, for example in Italy, Cyprus, Greece and Crete, have proved their longevity even under worse conditions over periods of 1500 to 3000 years [17].

Stochastics is the only means to evaluate the dispersion of the variety of parameters influencing the effectiveness of shaft sealing considerably. But, the necessary initial parameters, such as comprehensive statistic data, distribution and combination functions are lacking. If human failure is excluded by control and monitoring measures, safety assessment is necessary with regard to the following two questions with critical consideration of all influencing parameters:

- damage possibilities in the sealing and corrosion protection system.
- assessment of the loading cases and calculating procedures chosen for the abutment calculation.

The sealing and corrosion protection system consists of three parts.

- hydrostatic sealing above the abutment.
- slipping, sealing and corrosion protection layers at the abutment flanks.
- hydraulic sealing below the abutment.

All possible damage, such as sedimentation of the filler particles in the asphalt, asphalt losses due to penetration into joints, fractures and permeable areas, embrittlement due to physico-chemical effects, bitumen decomposition due to biological degradation, cracking in the slipping layer, impact of temperature increase, is excluded or restricted so that the entire system can not be endangered. Additional safety is provided by constructional design, dimensioning, material selection and redundancy of some elements.

Abutment and rock loading is determined with supposing unfavourable initial data. Thus, these loading conditions correspond to imaginary critical conditions, since the most unfavourable data for loading or loading combination, calculation, dimensioning and material parameters which represent safety limits were supposed. Therefore, an additional safety loading case is dispensible. The importance of shaft sealing in the multi-barrier concept of nuclear waste disposal justifies this suppositon of "simultaneous reach of the critical value of all influencing parameters". Despite these unfavourable suppositions, safeties of rock and abutment material have to be fixed with regard to the multi-axial strength-strain behavior. The respective values are essentially influenced by the chosen stress pattern (eg constant principal normal stress proportion, constant minimum principal normal stress, constant maximum principal normal stress, constant octahedral normal stress, determination of the minimum safeties) (Fig 9). If the expected stress pattern does not provide any data, it has been useful to make the minimum safeties the standard of comparison.

In the course of construction of the sealing until the end of repository operation the loading conditions change. The design of the entire construction presupposes the maximum loading in the starting period (eg development of the hydrostatic loading from above). In the course of operation load-reducing effects become stronger, caused by, eg rock pressure.development of fluid pressure from both above and below, cooling of the hot-emplaced asphalt, formation of elastic material behaviour in the material of the slip plane, concrete creep, concrete post-hardening and time-dependent inelastic material behaviour. On the basis of the here reported basic principles (slip planes between abutment and rock, application of sealing packages on clay and bitumen or asphalt basis) shaft and drive sealings have already been constructed for other purposes (gas storages in abandoned mine openings, sealing of flooded openings, shaft plugging) in different rock strata (even rock salt with inelastic strain behaviour) and are in operation for more than 15 years. For these purposes, cross section of about 100 m2 have been sealed against water pressure and brine pressure of up to 6 MPa. The constructions have always been absolutely impermeable. At present, sealing systems for a fluid pressure of 10 MPa are designed or in construction. Considering previous experience and presented considerations and recommendations (extra loading cases, sealing system, system of corrosion protection) it is possible to construct shaft and borehole sealings which fit the requirements of nuclear waste disposal to a large extent.

References

1. Sitz P. Querschnittsabdichtungen untertaegiger Hohlraeume durch Daemme und Propfen (Sealing of underground cross sections). Freiberger Forschungsheft A 163 (1982), VEB Deutscher Verlag fuer Grundstoffindustrie Leipzig.
2. Buettner G., Foerster S., Foerster W., Laube R., Sitz P. Die Abdichtung von Schaechten grossen Durchmessers als eine Voraussetzung fuer die Speicherung von Gas in stillgelegten Salzbergwerken (Design of sealing systems for shafts with large diameters—basis for gas storage in abandoned potash mines). In: Neue Bergbautechnik. Leipzig 4 (1974) 11, p. 836–839.
3. Sitz P., Bartl H., Geissler D., Hebestreit G., Herzel W., Schott E., Vorwerk W. Erfahrungen bei der Wiederinbetriebnahme der abgesoffenen Schaechte Bernterode I und Bernterode II. (Experience in reconstruction of the flooded shafts Bernterode I and

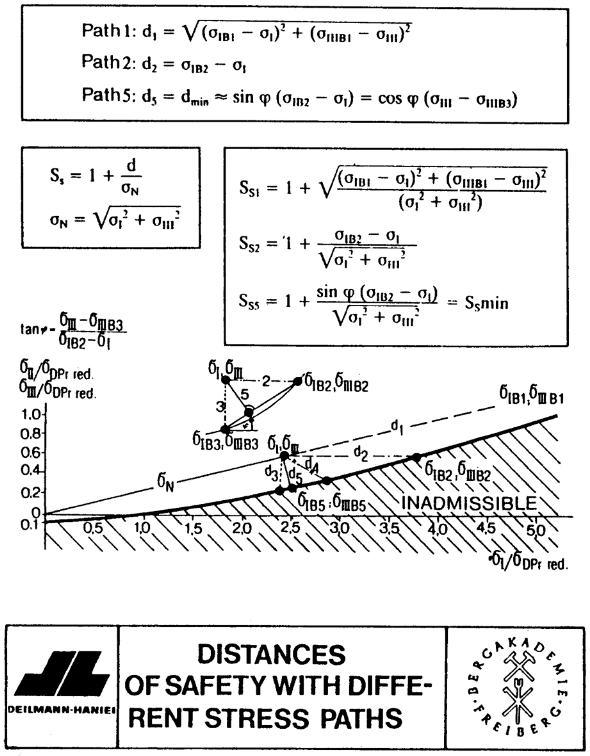

Fig 9

Bernterode II). Part I in: Neue Bergbautechnik 11. Leipzig (1981) 6, p. 352–356. Part II in: Neue Bergbautechnik 11. Leipzig (1981) 8, p. 437–442.

4. Arnold C., Sitz P. Die Entwicklung und Betriebsfuehrung des Stadtgasspeichers im stillgelegten Kalisalzbergwerk Burggraf-Bernsdorf (Development and operation of the town gas storage unit in the abandoned potash mine Burggraf-Bernsdorf). In: Freiberger Forschungsheft A 699 (1984), VEB Deutscher Verlag fuer Grundstoffindustrie Leipzig, p. 69–79
5. Oellers T., Sitz P. Entwurf und Berechnung gas- und fluessigkeitsdichter Schachtverschluesse (Design and calculation of gastight and watertight shaft seals). Special print from: “Schacht- und Tunnelbaukolloquium 1985” at the Institut fuer Bergbaukunde of the Technological University of Berlin/West.
6. Lersow M., Sitz P. Elasto-plastischer Deformationsansatz unter Verwendung der Fliessbedingung von DRUCKER/PRAGER fuer die Berechnung dickwandiger Rohre (Elastoplastic stress-strain relationships using the yield conditions given by DRUCKER/PRAGER for the calculation of thick-walled circular-cylindrical linings). In press in: Neue Bergbautechnik. Leipzig.
7. WP E 21 F/309 645 8 and P 38.38445.1. Hochbelastbare untertaegige Widerlager-konstruktion (High-load underground dam). Haefner F., Koeckritz V., Oellers T., Sitz P.
8. Haefner F., Koeckritz V., Sitz P., Voigt H.-D.Berechnung der Temperatur-Eigenspannung in massigen Betonbauwerken (Calculating the inherent temperature stresses on solid concrete structures). In press in: Bautechnik 65 (1988), Berlin/West.
9. WP E 02 D/313 374 5. Verfahren zur Reduzierung der Eigenspannungen infolge der Hydratationswaerme in massigen Beton-, Moertel—oder Zementbauwerken (Method for decreasing the inherent temperature stresses in solid concrete, mortar and cement structures). Sitz P., Haefner F., Koeckritz V.
10. Owen G.N., Scholl R.E. Earthquake engineering of large underground structures. Report N. JAB–000–128 1981, URS/J.A. Blume Associates San Francisco.
11. Dowding C.H., Rozen A.Damage to rock tunnels from earthquake shaking. Proceedings ASCE. Journal of Geotechnological Engineering. Div Vol 104 no. GT2, Febr p. 175–191.
12. Ichikawa Y., Shimizu H., Kawanobe M. Some dynamic behaviors of cavern walls during earthquakes. ISRM Symposium Aachen 1982. Rock Mechanics: Cavern and Pressure Shafts. Publishing House A.A.Balkema/ Rotterdam 1982, p. 307–320.
13. Fujita K., Haga Y., Ueda K., Nakazono K., Yoichi H. A seismic design of large caverns. ISRM Symposium Aachen 1982. Rock Mechanics: Cavern and Pressure Shafts, Publishing House A.A.Balkema/Rotterdam 1982, p. 267–274.
14. WP E 21 F/3155847. Verfahren zur Herstellung einer versiegelungsfaehigen Oberflaeche auf Carnallititgesteinen (Method for preparation of coatable surface of carnallitite rock). Krakau U., Sitz P., Wetzel D.
15. Nuessel H.Bitumen. Erdoelbuecherei vol 9, Huething und Dreyer GmbH. Mainz Heidelberg 1958.
16. Neuburger A.Die Technik des Altertums (Technology of the antiquity). Edition for Fourier Verlag GmbH. Wiesbaden 1986.
17. Langton C.A., Roy D.M.Longevity of borehole and shaft sealing materials. Proceedings of the Annual Meeting of the Materials Research Society. Boston Massachusetts, November 1983, 14–17.

Watertight lining systems to secure leaking shafts

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SYNOPSIS

Over 90% of the shafts still in use in the German Potash and Salt mines as well as a great number of shafts within the coal mines were sunk at the turn of this century.

In waterbearing formations these shafts were generally lined with cast-iron tubbings bolted together. Due to corrosion and mechanical damage, or material imperfections, inadequate installation and backfilling of the tubbing segments, a lot of shafts lost their watertightness and even part of their bearing capacity and stability. For safety reasons complete repair became necessary. Moreover, the necessary permanent shaft repair work was expensive and caused loss of hoisting capacity in production shafts.

Lining systems had to be found which could offer watertightness and bearing capacity to today's standards, together with a minimum on future repair and shaft inspection work.

Owners and shaft sinking contractors together established design criteria and developed concepts of watertight lining systems to be installed into existing shafts. Those concepts are mainly based on today's shaft lining techniques in waterbearing formations and the lining techniques used in pressure tunnels of hydro-power plants.

Meanwhile 16 shafts with a total length of about 2.900 m have been relined during the last 20 years, no less than 14 shafts since 1980.

Criteria and concepts for new watertight lining systems, their advantages and disadvantages are described below. The planning and installation of a new watertight lining for a ventilation shaft and a production shaft are shown.

INTRODUCTION

Shafts are not only the main entrance to the orebody, they are in general the way for the miners working underground to escape. Therefore they have not only an operational function, but also a safety function of vital importance for miners as well as owners.

During sinking, the shaft is normally lined immediately according to the type and behaviour of the formation. Formations which are waterbearing, stable or unstable and running are considered as critical formations not only during sinking but also when the shaft is in use. In running formations, shafts are not only endangered by water inflow but also by material inflow which can cause loss of stability and bearing capacity.

For each type of mining whether ore, coal or potash, the bearing capacity and stability of the shaft lining is of vital importance. Watertightness, however, is far more important for a salt mine than for any other kind of mining because of the soluble character of salt rock. Minor water inflows—dangerous for salt mines—can be handled in ore and coal mines mostly without problems.

Up to the early fifties, when single and double welded or riveted steel ring-concrete composite linings were introduced (later on followed by completely welded steel liners), waterbearing formations were generally lined with cast-iron tubbings bolted together.

While installing the tubbing segments, dimensional inaccuracies, material imperfections, inadequate bolting, installation and backfilling already caused reductions in bearing capacity and stability from the beginning. Formation movements and changing temperatures (summer-winter; incast-outcast) led to varying degrees of loading on the tubbing.

Aggressive formation water, ventilation air humidity together with salt dust caused intensive corrosion and reduction of wall thicknesses. Annual cor rosion rates in excess of 0.1 mm were observed. Loss of backfill by seepage or erosion caused loss of resistance to bulging and buckling. Even repairs such as backfill or sealing injections can cause locally non-uniform loads.

In the long run the tubbing lining can lose watertightness, stability and bearing capacity and the shaft as a safe way out is endangered. Financial loss arises from the permanent repair work, in addition to production losses caused by reduced hoist capacity.

A complete overhaul was more or less inevitable for a part of the more than 60 year old tubbing lined shafts. In 1968, a 316 m long new lining was built into a salt mine shaft for the first time. The mine concerned had finished production and was being rebuilt as a nuclear waste repository test site for which a higher safety level was required. In 1976, the production shaft of a potash mine which had partially lost watertightness and bearing capacity was provided with a 231 m long new lining.

A sudden, heavy water inflow in a minor ventilation shaft led Kali und Salz AG to investigate and evaluate systematically all (39) shafts in use.

Estimating the effective bearing capacity as well as the quality of the concrete backfilling appeared to be—as expected—difficult. In order to protect their shafts on a long term basis against sudden water inflows, to improve watertightness and structural support to today's standards, as well as to reduce costs for shaft repair and inspection to a minimum, Kali und Salz AG decided to secure a number of shafts by installing a new watertight lining.[1] Since 1980, a total of 14 shafts have been secured with watertight linings up to a total length of 2.346 m and single lengths up to 280 m. Only in one case was a coal mine shaft involved. This once more shows that although coal mine shafts are much more affected by formation movements, watertightness is far more important in salt shafts.

CRITERIA AND CONCEPTS

Lining systems have been developed to secure older leaking and/or unstable shafts.[2]

These lining systems have to be watertight and able to withstand all external pressures. They must prevent penetration of formation water into the shaft below the new lining and the dead weight of the new lining has to be transferred to the rock.

Principally, the following components can be used to meet the various requirements:

structural support:
reinforced concrete, steel liner or a combination of both as well as prefabricated segments;

- watertightness:
  asphalt layer, fully welded steel liner, or fully welded plastic sheet;
- sealing system:
  asphalt layer with a liquid pressure greater than the hydrostatic formation water pressure, chemical-seal-ring or by grouting;
- load transfer:
  direct connection of the lining system to the existing lining or placing a concrete foundation ring at the end of the new lining.

A total of 16 new watertight linings has already been installed. Final conclusions can thus be drawn and justified by the large number of completed projects.

Three systems were generally adopted for technical and economical reasons (Fig. 1):

System 1:

Reinforced concrete cylinder with an outer fully welded steel liner and asphalt layer;[3]

System 2:

Jointless reinforced concrete cylinder with asphalt layer;

System 3:

A fully welded steel liner with concrete backfill and a chemical-seal-ring.[4]

According to these systems 14 of a total of 16 new watertight linings were installed.

Four shafts (total length 881 m; formation water pressures up to 35 bar, single length up to 316 m) were secured according to system 1. Six shafts (total length 925 m, formation water pressures up to 21 bar, single length up to 185 m) according to system 2 and 4 shafts (total length 526 m; formation water pressure up to 55 bar, single length up to 231 m, wall thicknesses up to 65 mm) according to system 3.

Structural support for the systems 1 and 2 is provided by the reinforced concrete cylinder, for system 3 by the relatively thick-walled steel liner combined with the concrete backfill.

The watertightness in horizontal direction of system 1 is secured by the asphalt filled annulus between existing and new lining in addition to the fully welded steel liner. For system 2 water-tightness results from the asphalt in combination with the

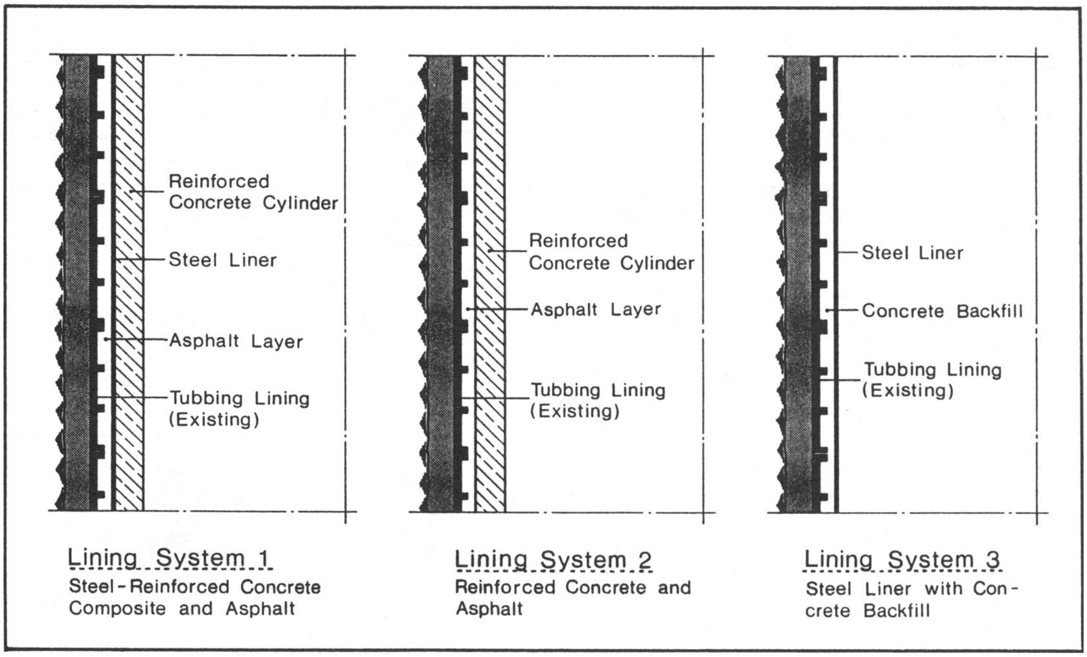

Fig.1 Systems used for new Watertight Shaft Linings

jointless concrete cylinder which must in turn be asphalt-tight. In system 3 the fully welded steel liner alone provides watertightness.

Watertightness in the annulus between old and new lining in vertical direction downwards to prevent seepage along the foundation is provided for systems 1 and 2 by the asphalt. For system 3 a chemical-seal-ring was installed.

The dead weight of the new lining in systems 1 and 2 (with a gliding asphalt layer!) has to be transferred to the rock by a foundation ring dimensioned accordingly. In system 3 the load is transferred to the backfill mortar and further on to the existing lining or to the rock by slip bolts arranged at the outside of the steel liner.

Structural support of the concrete cylinder for system 1 and 2 is designed according to the "Guidelines for Water-tight Shaft Linings",[5] on the assumption that the existing formation load is—as before—absorbed by the existing lining. The steel liner in system 3 is designed according to the principles of Amstutz/Link.[6] The rigid concrete backfill is essential in this system. An open joint between steel liner and backfill has an essential influence on the bulging safety of the steel liner. An open joint can be caused, for example by temperature changes in the downcast fresh air. This has to be taken into account in the design.

The two first systems are directly developed from shaft lining in water-bearing formations. System 1 represents the standard lining for freeze shafts in West German mining since the sixties. System 2 is also used for conventional shaft construction in stable, slightly to moderately waterbearing formations with water pressures of up to 10 bar. System 3 was developed from pressure tunnel construction for hydroelectric power stations.

Other systems with structural elements such as prefabricated concrete segments, cast-iron or steel tubbings and with sealing elements such as plastic sheets or clay mixtures have not been adopted. They are not only technically but also economically inferior to the mentioned systems with regard to the market situation in the Federal Republic of Germany. The sealing problems at the numerous connection points of the prefabricated elements are basically the same as with the former tubbing linings. The tightness of plastic sheets, especially at the welded points, is unreliable particularly with higher water pressures.

Choice of lining system

Although quality characteristics for watertightness and structural support of the three systems are nearly identical, they differ considerably, however, for the mine owner and user on other important criteria. These criteria are the total wall thickness of the system and the total costs.

With a formation water pressure of for example 15 bar, the total wall thickness of system 1 and 2 is appr. 40–50 cm, of system 3 only 10–15 cm. For potash shafts with a diameter of about 5 m this means a diameter decrease of appr. 80–100 cm with system 1 or 2, a loss of about 30–35% of the cross section. With system 3, however, the loss is only about 5%. In general, the diameter loss is justifiable in minor shafts or pure ventilation shafts, more so because it has been proved that using the same ventilation equipment the volume of air is even increased by reduction of air friction. In production shafts, the total cross section is generally required and a further reduction can hardly be accepted.

Also the total costs (including all installations, mobilization and demobilization as well as the necessary special equipment such as the suspended working stage, ropes, cables and similar) vary considerably. Related to the lowest-priced system 2 (100 %), the total costs for system 1 amount to appr. 150–200% and for system 3 to appr. 250–350%.

The customers' choice is therefore clearly directed. Wherever a greater diameter reduction can be accepted, the relatively cheap new lining system 1 will be used. System 2 has to be applied in case of formation water pressures higher than appr. 20 bar. Where reductions can hardly be accepted for cross section reasons, the more expensive system 3 must be used. Up to now all new lining systems according to system 3 have only been installed in production shafts.

PROJECTS EXECUTED SHAFT RÖSSING-BARNTEN

Planning and design

Since 1983, the shaft Rössing-Barnten part of the Siegfried Giesen potash mine owned by Kali und Salz AG—has been used only as a downcast ventilation shaft of minor importance. The shaft was sunk through the waterbearing formations of the overburden and the gypsum cap rock to a depth of 120 m by freezing; it was lined into the competent salt rock up to a depth of 150 m with cast-iron tubbings and with concrete within the salt rock.

The partly corroded and below appr. 120 m depth leaking tubbing column was to be secured and sealed, so that for the future practically no further shaft repair work and inspection would be necessary. This was the prerequisite for the planned disassembly of the existing hoisting installation, the shaft furniture and the surface installations. The shaft Rössing-Barnten has an inner diameter of 5 m. Due to its minor importance in the ventilation system a significant decrease of the diameter could be accepted.

The new lining system was to be based in the competent salt rock, where seepage could be excluded. Control holes drilled through the shaft wall showed an appropriate foundation depth at 164 m.

Choice of the lining system

The lining system No. 2 was chosen—Reinforced concrete cylinder with sealing asphalt layer (Fig. 2). To limit the diameter reduction and to save asphalt, the asphalt annulus was to be as small as possible. Therefore a permanent outer formwork had to be provided, for which liner plates with a wall thickness of 2.7 mm were chosen. The tubbing column was only slightly out of round, so that for the asphalt annulus only appr. 7 cm were sufficient.

Structural design

The concrete cylinder was designed on a uniform load out of the hydrostatic pressure of the asphalt with a specific weight of Y=1.3 Mp/m^3. Irregular pressure was not considered. On the one hand no mining influences were imaginable, on the other hand it could be supposed that the existing shaft lining would be able to support as before the formation pressure and its possible irregular part. The structural design of the concrete cylinder resulted in a wall thickness of 34 cm and a remaining shaft diameter of 4.08 m. The concrete cylinder with a concrete quality of B 25 and B 35 had to be reinforced only in the lower 20 m.

Foundation and sealing system

The total dead load of the new lining system and the asphalt layer rests on a foundation ring. The reinforced foundation was designed in a circle-shaped way, 2.20 m high with an average width of 1.30 m and with a support ring of usual construction (Fig. 3). The dead loads were distributed between the existing concrete shaft lining and salt rock. The sealing system consists of the asphalt with full hydrostatic pressure in direct contact with the competent rock and a sand asphalt layer with low plasticity. The asphalt seals the annulus at the bottom. The sand asphalt does not allow a further seepage of asphalt into the foundation area. In addition a gravel layer with grout pipes was arranged, which could be injected if necessary. Grout pipes were also arranged in the foundation ring.

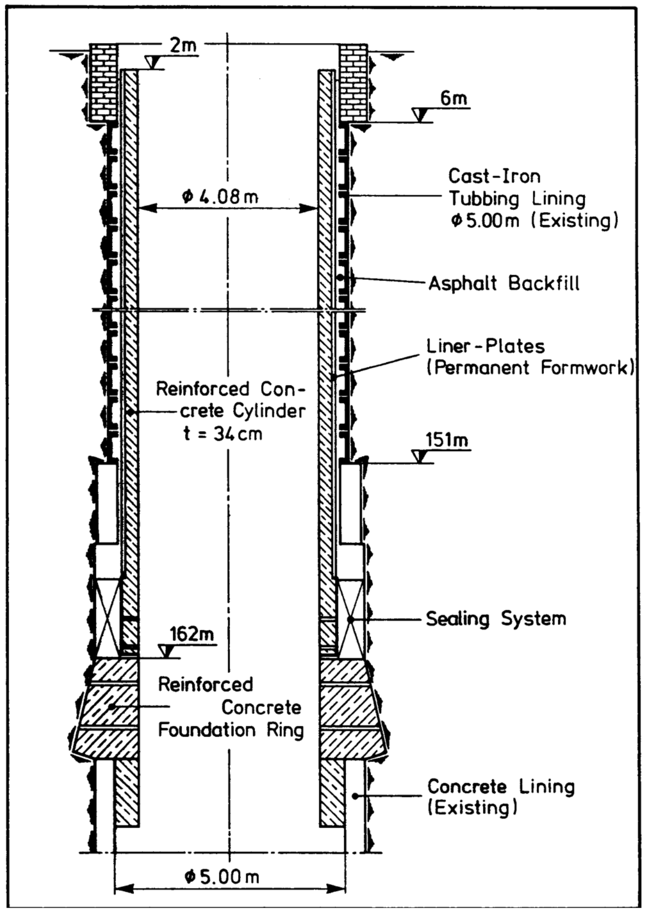

Fig. 2 Shaft Rössing-Barnten New Watertight Lining System

Asphalt annulus

The asphalt was to be filled in the annulus in the concrete-lined shaft section below the tubbings as soon as possible to prevent the washing-out of salt rock directly above the sealing system. On the other hand the tubbing section was only to be backfilled after full completion of the column due to safety reasons—heat stress on the tubbing column. For this, special pipes suspended into the annulus were provided.

Construction of the jointless monolithic concrete cylinder

After detailed consideration, it was decided to build the jointless monolithic concrete cylinder with the help of a climbing formwork, consisting in total of 5 formwork rings, each 1.4 m high. For the construction of a jointless concrete cylinder it is necessary to pour the concrete continuously in layers and compact the individual layers by vibration, as is done for example when using slipforms. With a climbing formwork the individual formwork rings have to be moved from bottom to top so fast that the concrete of the last section has not hardened before the next concrete layer is poured. The climbing formwork must be elongated continuously in advance of the pouring of the concrete (Fig. 4).

This was found to be feasible. In case of interruptions the installation of a construction joint tape was provided.

Some technical and economical advantages were offered by this method:

- The necessary installations, such as the working stage, were available and used to draw off the shaft furniture.
- The assistance of a slipform construction subcontractor was not necessary.
- With a total formwork height of 7 m and concrete setting times of appr. 12 hours, pouring of 12 m concrete lining per day was possible, that is significantly quicker than applying slipforms.
- Fissures which can occur when concrete adheres to the formwork during sliding and which can cause leakages after setting are avoided when using a climbing formwork.

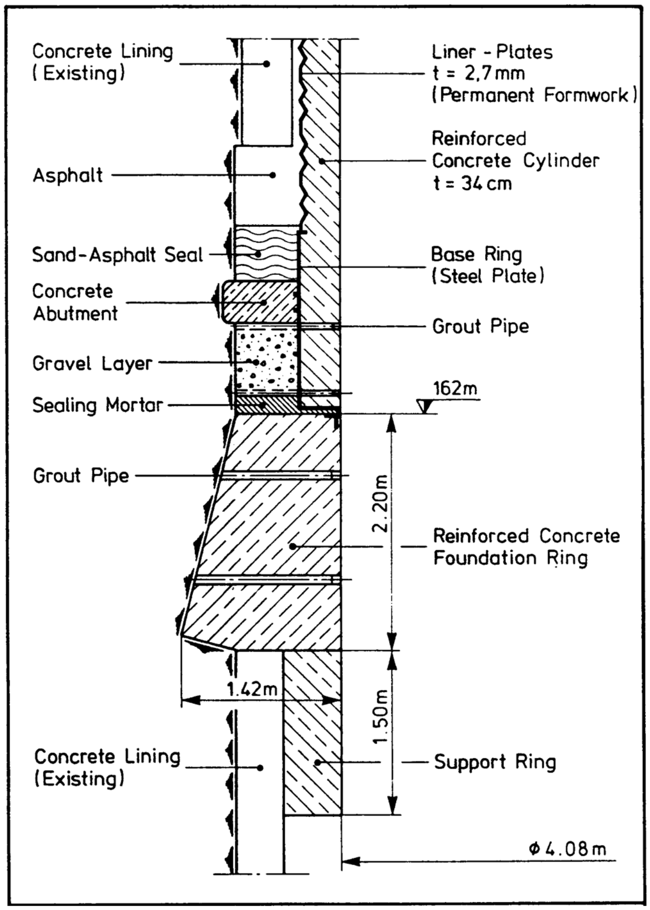

Fig. 3 Shaft Rössing-Barnten Foundation and Sealing System

Execution of work

Work started in May 1984 with the dismantling of the hoisting installations; the shaft furniture had to be drawn off. To carry out this work a two-deck-working stage was installed. For hoisting, the existing Koepe hoist was converted into a drum hoist. Buckets were used for transport of material and manriding. A small concrete mixing plant was installed near the shaft on the surface.

To install the new watertight lining the two-deck working stage was completed with a third—hanging—deck. The upper (top) deck was used for suspension, the installation of the outer permanent formwork (liner-plates) as well as for discharging the concrete. The middle deck was used to install the reinforcement, distribute the concrete and set and align the formwork rings. The third deck was situated at the level of the lowest formwork ring. This ring was loosened parallel to the concreting and was lifted to the middle deck. To set and align this ring, concreting had to be interrupted for a short time.

For pouring the concrete cylinder with a length of 160 m appr. 15 days were needed, the average pouring rate being 10.5 m/d, the maximum rate 13 m/d. The work could be executed successfully under nearly ideal circumstances without being under deadline pressure.

SHAFT SALZDETFURTH 1

Planning and design

The downcast shaft Salzdetfurth 1 is the main production shaft of the Salzdetfurth potash mine owned by the Kali und Salz AG (Fig. 5). In the waterbearing formations of the bunter sandstone and top Zechstein-clay between the depths of 52 to 192 m this shaft is lined with cast-iron tubbings. Its diameter is generally 5.25 m except between the depth of 144.7–149.2 m. In this depth a socalled steel collar with a diameter of 4.80 m had already been installed to secure some damaged tubbing segments. The shaft is situated on a hill, the groundwater level is found at a depth of appr. 50 m. In order to avoid water inflows from the lower part of the Zechstein-clay the new watertight lining was to extend into the competent rock of the younger Na4

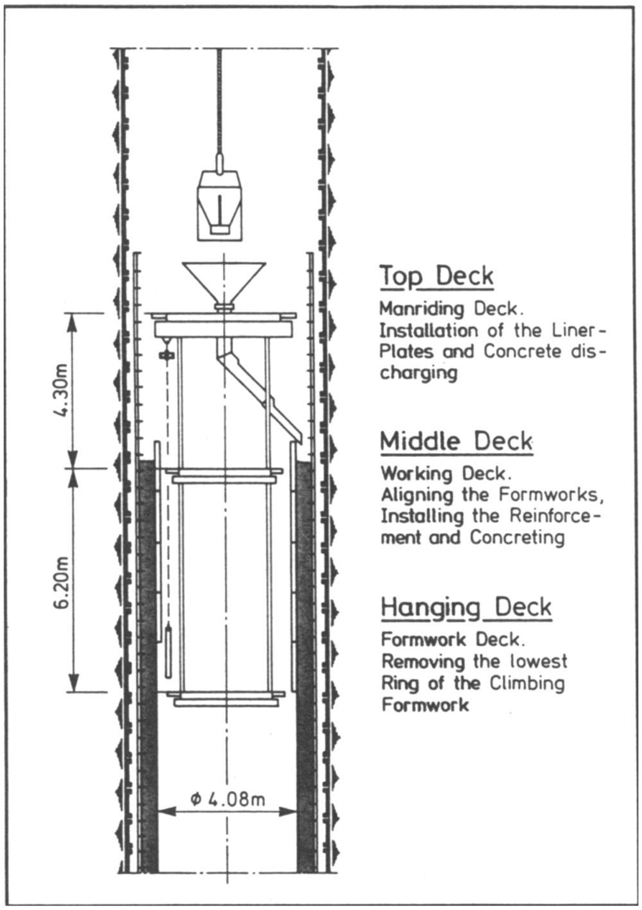

Fig. 4 Shaft Rössing-Barnten Working Stage

Rocksalt formation. This resulted in a foundation depth of 227 m and a total length of the new lining up to the groundwater level at 50 m of 177 m.

The shaft is equipped with a double skip production hoist, an emergency hoist, some cables and pipelines as well as shaft furniture (shaft buntons, guides etc.). The new watertight lining though being without foundation and sealing system—had to be installed in a maximum of 6 weeks shutdown (holiday) period in the summer of 1985. During the production stop for Christmas holidays 1984/85, 16 days were available to install the foundation ring and sealing system. Thus all the planning parameters were known. The construction period of 6 weeks compared to the length (177 m) of the new lining was extremely short.

Choice of lining system

The double skip hoist was set up to the diameter of the steel collar (4.80 m). As the removal of the steel collar was out of question for reasons of safety and construction time, any further diameter loss had to be kept to a minimum. Therefore, as a new watertight lining, only a concrete backfilled steel liner according to system 3 was feasible (Fig. 6). The maximum possible inner diameter of the steel liner proved to be 4.55 m.

Structural design

Structural design was based on the hydrostatic pressure of the saliferous formation water being in the lower end 2065 kN/m^2. Temperature changes in the downcast shaft were taken into account in design calculations by a 1.5 mm open joint between the steel liner and the concrete backfill, thus with a somewhat reduced bedding. Including 2 mm reserve for corrosion, design calculations resulted in a steel wall thickness of 16 to 45 mm (steel quality St.E 36 according to German standards) and a total weight of appr. 690 t. As a rule, the wall thickness of each 6 m section from bottom to top was reduced by 1 mm. The dead weight of the steel liner could be transferred by slip bolts at the outside of the steel liner to the concrete back-fill and thus to the existing shaft lining. So only a small foundation ring was found to be necessary.

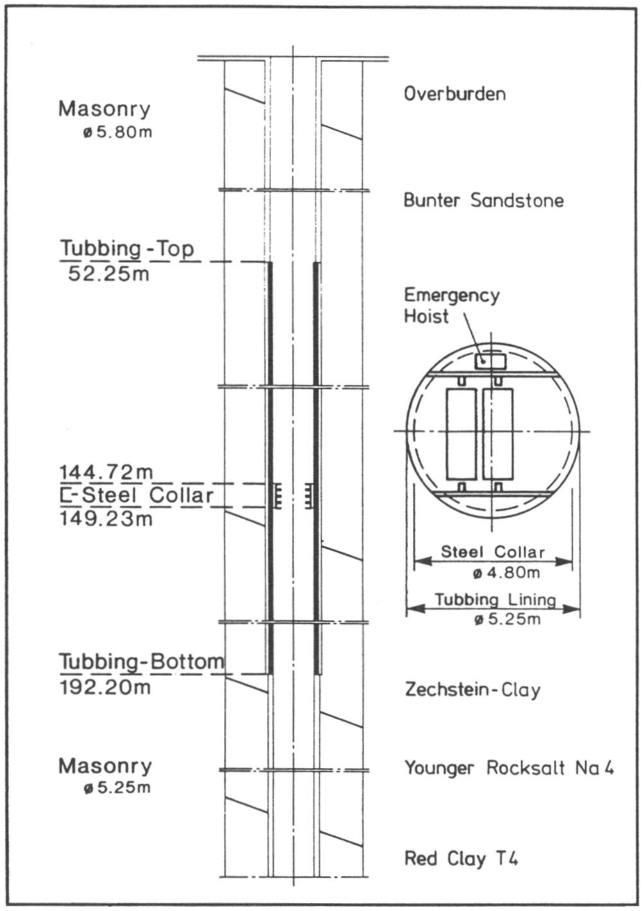

Fig. 5 Shaft Salzdetfurth 1 Existing Lining, Geology, Cross Section

Foundation and sealing system

Foundation ring and sealing system are integrated to form a single unit (Fig. 7). To construct the foundation ring the existing shaft lining (masonry) had to be removed. It is important to remove the existing lining in the sealing zone completely up to the solid rock. For sealing, a synthetic material —i.e. Dowell-Chemical-Seal—is used. This material is poured as a liquid and hardens afterwards to form a rubbery compound. As soon as water is added it swells. In this way a chemical seal or packer is created. The steel caisson at the base of the liner serves to reduce the volume of the seal compound and at the same time as an abutment for sealing injections.

The planning stages described so far refer to the new lining system as a construction whereas considerations of the necessary construction time had already to be included. The following planning stages refer to the installation of the new lining. Construction time was given absolute priority.

Steel liner

From earlier projects it was already experienced that the construction time is related very closely to the length of the single sections. In order to have any chance at all to install the steel liner completely within 6 weeks, the sections had to be as high as could possibly be handled. Detailed examination of the working conditions in the shaft hall, the guide frame as well as in the shaft, showed that handling 6 m sections would be possible. For reasons of space each ring section had to be divided into 5 segments with different circular measurements.

Work on a parallel basis

In order to keep to the strict completion schedule, many activities had to be carried out simultaneously (Fig. 8). However, the following single activities could only be executed separately for operational and safety reasons:

– moving the working stage,

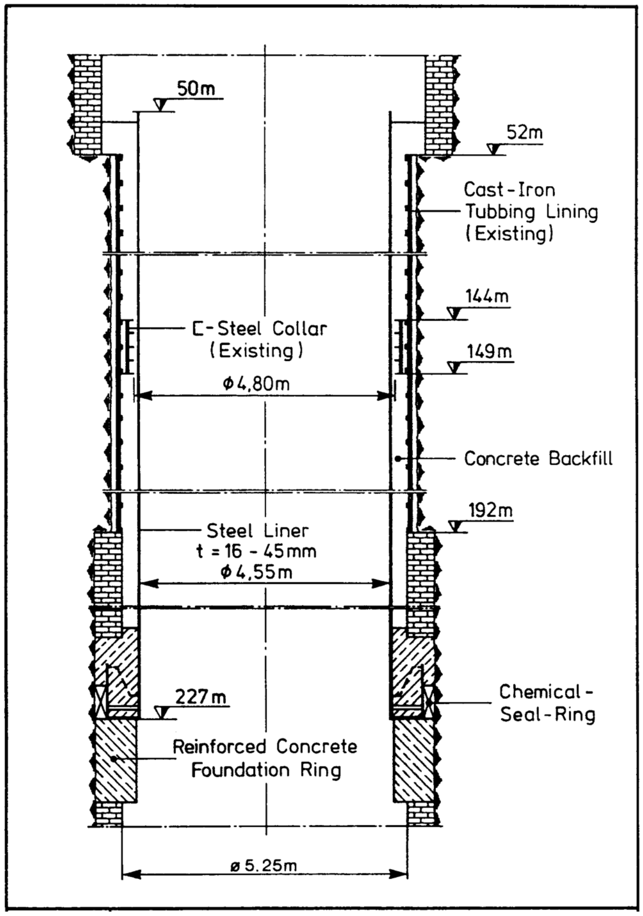

Fig. 6 Shaft Salzdetfurth 1 New Watertight Lining System

– removing the buntons for the next section, and
– lowering the segments and adjusting their position.

All other activities such as aligning, assembling, welding the vertical and horizontal joints, ultrasonic inspection of the welds as well as reinstallation of the buntons and backfilling with concrete had to be done simultaneously.

Working stage

The fact that so many activities had to take place at the same time meant that a multi-deck stage with 6 m deck distances determined by the section height was required (Fig. 9). Detailed considerations and analysis of the working method and the sequence of operations resulted in the following arrangements:

– two decks (6 and 5) for aligning the segments and welding the vertical joints,
– two decks (4 and 3) for welding the horizontal joints,
– one deck (2) for ultrasonic inspection of the welds and repair, if necessary, and
– one deck (1) to reinstall the buntons, fix the guides and install the new cable retainers.

Furthermore, a top deck for suspension and manriding and four intermediate equipment decks had to be installed, resulting in a total length of 36 m.

For safety reasons (fire hazard), shaft examination must be possible at any time. So an opening for the skip used for manriding to pass the working stage had to be provided. That is why two separate stages with openings (which were closed with traps during normal operation) were used. Including working load both stages together had a total weight of 36 t. This very extensive stage system was a necessary requirement in order to meet the time schedule.

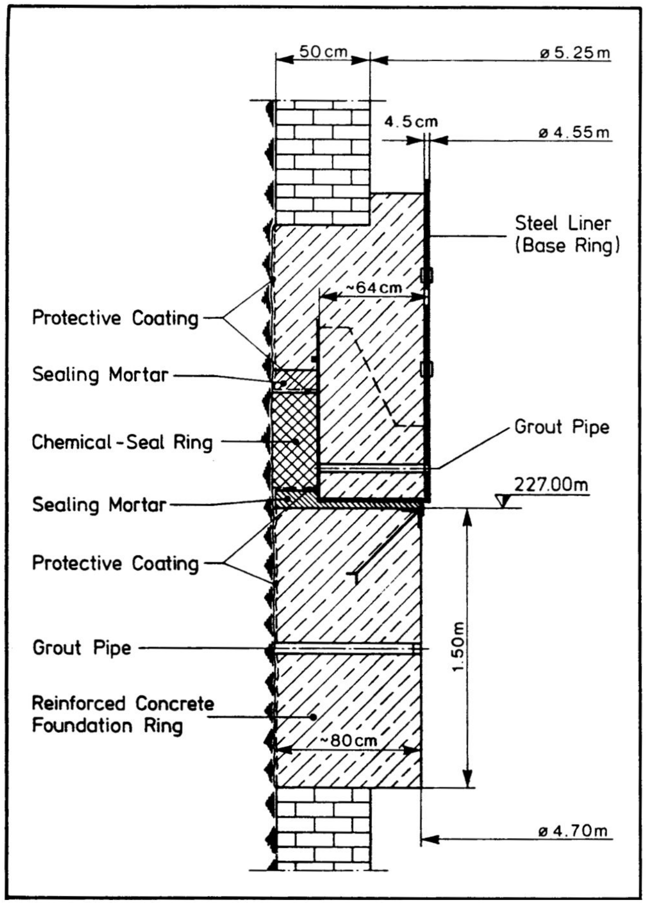

Fig. 7 Shaft Salzdefurth 1 Foundation and Sealing System

Concrete Backfill

The following requirements were made on the backfilling material as rigid bedding for the steel liner:

- it had to be pumpable and self-levelling without compaction by vibrating,
- filling out absolutely all cavities,
- without sedimentation during setting, as well as
- a compressive strength of at least 15 N/mm^2.

The backfill concrete was especially developed for this purpose. It consisted of sand with a grain size of less than 2 mm, cement as well as liquefying and stabilizing chemical additives, and with a water/cement factor of 0.59. The concrete was delivered to the job site readymixed.

Preparatory work

Scope and execution of the necessary preparatory work during production were also fixed in detail within the planning phase. These were for example the removal of shaft furniture (as far as possible), cleaning the shaft walls, installation of support beams for ropes and cables, dismantling of the existing shaft covering and installation of a provisional sliding platform as cover, the installation of brackets for provisional working platforms and support beams as well as the erection of the surface installations.

Execution of work

The reinforced concrete foundation ring together with the first 2.0 m liner section and the sealing system were completed during the Christmas holidays 1984/85 within 12 days. This was done with temporary provisional working platforms. The

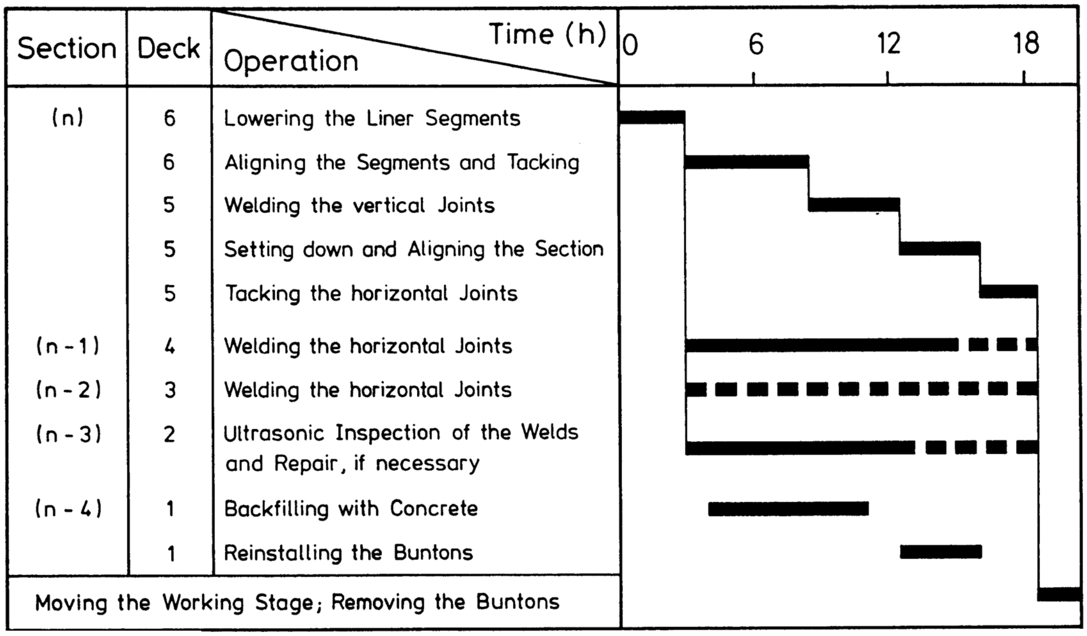

Fig. 8 Shaft Salzdefurth 1 New Lining-Sequence of Operations

exposed salt rock was solid and dry. The pumpable concrete and chemical-seal-compound was transported into the shaft by downpipes. Automatic welders were already used.

There was a time span of 43.5 days for the installation of the new steel liner. The assembly time in the shaft had been fixed in the time schedule at 5 days according to previous experience. This allegation, however, proved to be too optimistic. The assembly of the two working stages with a height of 36 m for which a prefabrication was only possible to a limited degree with regard to the existing shaft furniture and hoisting installations required much more time than expected. Also the installation of all welding equipment with accessories on the 6 main decks and 4 equipment decks as well as all electrical equipment turned to be far more complicated. Altogether some 3 days more were required. However, there were good chances to compensate for this delay.

Before reporting on the further sequence of operations, the working cycle for one single section should be described (Fig. 8):

1. Lowering the single segments of one section one after another into the shaft, setting them down on rollers running on top of the previous section and then moving them into final position.
2. Aligning the single segments and tacking them together.
3. Turning the whole section by appr. 36 degrees to the fixed installed automatic welders, welding of the vertical joints, aligning the section and tacking the horizontal joint.
4. Welding the horizontal joint on the No. 3 deck and completing the previous horizontal joint (as far as required) at the No. 4 deck with semi-automatical welders.
5. Ultrasonic inspection of the welds and, if necessary, repairing them on the No. 2 deck.
6. Reinstalling the buntons on the No. 1 deck.
7. Backfilling with concrete.
8. Moving the working stage by 6 m and removal of the buntons for the next section.

Welding was done with inert gas. To prevent interruptions of the welding operations, shaft ventilation was limited to about 1000–1500 m^3/min. The backfill concrete was pumped on surface to the suspended downpipe and delivered behind the steel liner. Backfilling was done for every section up to a level just below the last uninspected horizontal weld. A total quantity of 1000 m^3 was necessary.

The planned sequence of operations and the organization of work proved to be successful. No changes were necessary nor were principal new aspects for the future experienced. However, there were some outside influences which created considerable difficulties.

As in other cases, the tubbing lining leaked and the shaft had to be characterized as humid to wet. Particularly humidity can produce considerable difficulties during welding operations. An effective protection cannot be installed. Furthermore,

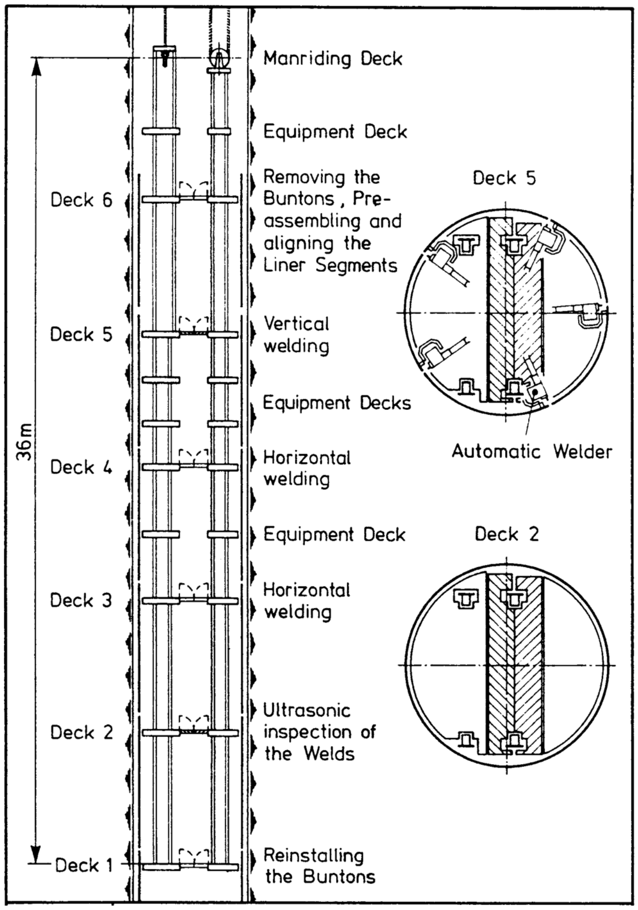

Fig. 9 Shaft Salzdefurth 1Working Stage

additional water due to heavy rains was dripping into the shaft. Together with salt dust the wet atmosphere caused significant disturbances in nearly all electrical devices and control elements of the welding equipment. Standstills and delays resulted.

As the comparison of planned and actual construction times shows significantly, the delays due to the above-mentioned problem amounted to appr. 70 hours within the first 10 days up to section No. 6 (Fig. 10). Only then were the planned construction times per section realized. These delays however could be made up during installation of the remaining sections (Nos. 7–31). The delays resulting from the assembly work could not however be compensated completely. It should also be mentioned that a deplorable fatal accident occurred caused by an object falling down the shaft.

At the end there was a delay compared with the planned time schedule of appr. 40 hours. Despite this delay which caused some rearrangements for the Salzdetfurth mine, the installation of this fully welded 177 m long steel liner in such a short time is considered by the owners and the contractor to be a complete success. It was realized under extremely difficult conditions and was only possible due to the great commitment of all parties concerned.

Acknowledgement

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References

1. Potthoff A.: Überwachung, Sicherung und Reparatur von Schächten der Kali und Salz AG (Controlling, Securing and Repairing Shafts of the Kali und Salz AG). In: Kali und Steinsalz 9 (1986), Nr. 7, p. 213–222.
2. Brune H.; Schauwecker E.; Sicherung alter Tübbingschächte durch wasserdichte Vorbausäulen (Securing of Old Tubbing Shafts by Installation of a New Watertight Lining System). In: Glückauf 119 (1983), Nr. 20, p. 993–997.
3. Stoß K.; Braun B.: Installation of a Watertight Lining to Secure a Leaking Salt Shaft. The First International Potash Technology Conference, Saskatoon, Canada.
4. Bergmann K.; Link H.; Schauwecker E.: Einbau einer Vorbausäule in den Schacht Sigmundshall (Installing a New Watertight Lining System in Shaft Sigmundshall). In: Kali und Steinsalz 7 (1979), Nr. 9, p. 379–388.

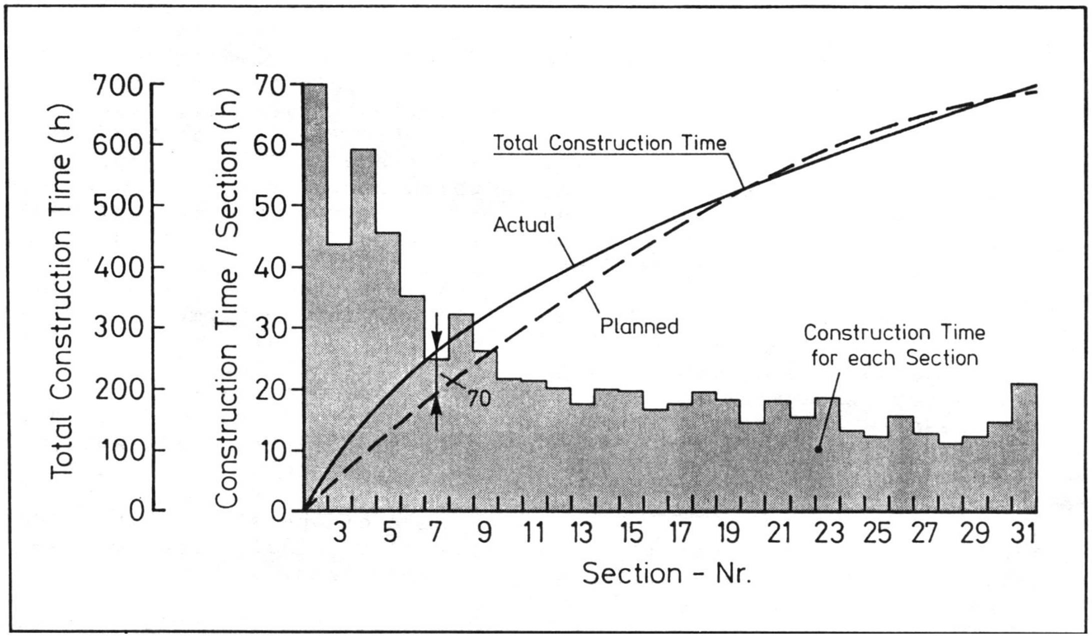

Fig. 10 Shaft Salzdefurth 1 New Lining-Construction Time

5. Link H.; Lütgendorf H.O.; Stoß K.: Richtlinien zur Berechnung von Schachtauskleidungen in nicht standfestem Gebirge (Guidelines for the Structural Design of Shaft Lining Systems in Unstable Rock). Published by Steinkohlen-Bergbauverein, Verlag Glückauf GmbH, Essen, FRG.
6. Link H.: Zur Bemessung dünnwandiger Vorbausäulen in Tübbingschächten (Designing New Steel Liners for Tubbing Shafts). In: Glückauf-Forschungshef te 47 (1986), Nr. 5, p. 244–251.

Review of developments in precast concrete shaft linings in the United Kingdom

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Synopsis

Standard bolted precast concrete linings have been in use for many years in both tunnel and shaft construction. Since their introduction some fifty years ago, they have grown to become the standard method of sinking and lining a shaft in soft ground conditions in the United Kingdom.

In the last few years there have been a number of developments in the standard linings and new types of lining have been introduced.

This paper gives a review of traditional bolted concrete shaft linings and shaft sinking techniques and then reviews the developments and new lining types with examples of their current use in the U.K.

The new developments covered include:

The use of rubber compression gaskets and hydrophilic rubber sealants to prevent water ingress.

The design and erection of smoothbored precast shaft linings allowing a single lining operation, including details of a 75 m deep shaft in soft ground conditions.

Large diameter shafts used for pumping stations up to internal diameters of 35.5 m.

Specialist designs and applications including storage bunkers and spiral chutes at great depths in British coal mines.

The range of items ancillary to shaft construction that are now available in precast concrete, for example roof and landing slabs, support rings and corbel rings.

STANDARD BOLTED LININGS

Standard Bolted Segmental precast concrete linings are an integral part of modern underground construction. Over 40 different sizes are now available from the manufacturers, ranging in diameter from 1.52 m I.D. (Internal Diameter) up to 38.0 m I.D.. In shaft applications they can be regarded as providing the finished permanent lining, the primary lining only or they may just be considered as a method of construction.

The standard rings may be used to line either shafts or tunnels; generally the smaller diameters (Up to 4.0 m) are used for tunnels and the larger diameters for shafts, and although similar the linings for shafts and tunnels developed separately.

Precast concrete tunnel linings evolved first; a smoothbore rapid lining was developed in the 1930s[1] and the first true reinforced concrete bolted tunnel lining was designed by Mott, Hay & Anderson in 1939[2]. This lining was generally similar in form to the cast iron lining it was designed to superceed. A standard range of bolted concrete tunnel linings was introduced by the tunnelling contractor and precast manufacturer Kinnear Moodie Limited after the war, but it was not until 1948/49 that they developed the first precast bolted linings for shaft sinking. At that time most of the tunnelling and shaft work in the country was centred on main drainage projects in and around London and the precast shaft linings were designed to replace timbered shafts.

For many years the standard shaft lining size was 12'-00" I.D. (3.66 m) The standard main drainage tunnel at that time used a 6'-5" I.D. (1.96 m) precast ring lined with insitu concrete. The 12'-00" I.D. circular shaft just allowed the introduction of the 7'-6" diameter tunnelling shield required.

A typical shaft ring of 3.66 m I.D. consisted of four ordinary segments, two slightly shorter top segments and a small solid key or closure piece. Each ring was usually 610 mm (2'-0") wide, (See fig 1). The segments are bolted to each other circumferentially to form a ring, two bolts connecting each segment. The key (stone) is provided to allow the placing of the last segment in a ring and is the final piece to be inserted, it is parallel sided.

It was not until the late 1950's that shaft segments started to be used more widely and gradually the number of different sizes available for shaft linings grew as they were developed for individual projects. The popular sizes were 10'–0" (3.05 m), 11'–0" (3.35 m), 12'-0" (3.66 m), 13'-0" (3.96 m) 14'–0" (4.27 m) and 15'–0" (4.57 m I.D.). As the range of sizes increased it became uneconomic and inconvenient to produce separate tunnel and shaft linings of the same diameter and, with the

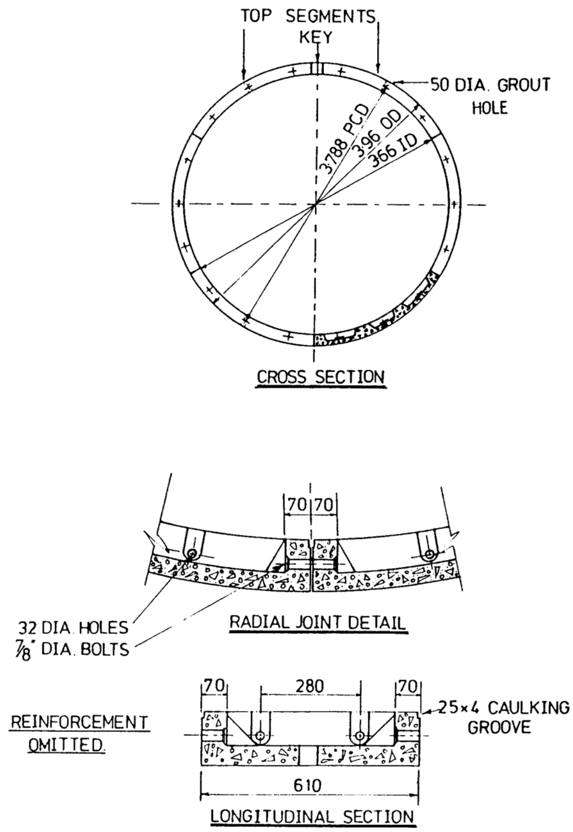

Fig 1. Typical standard bolted lining

exception of some common sizes, the smaller diameter linings were designed to serve as either—so that now there is a completely integrated range available, (fig 2).

Unlike tunnel linings, linings for shafts did not have to cater for the large stresses induced by the shield rams during tunnelling nor did they have to cope with varying vertical and horizontal ground loads; a uniformly distributed radial ground loading could be assumed. The result was a lighter and hence cheaper lining that was easy to transport and build.

The standard linings are designed to suit the range of ground loading normally encountered in U.K. practice; the common shaft sizes are normally satisfactory to depths of 20m. The smaller diameters are proportionately stronger and have been used at depths approaching 40m, whilst the larger diameters are rarely required at depths greater than 20 m. Allowable lateral ground pressures vary between 200 KN/m^2 to 700 KN/m^2 depending upon the diameter and the manufacturers are happy to advise on their use in particular situations.

SHAFT SINKING TECHNIQUES

The reason for the eventual adoption by the whole of the industry lies in their economy and ease of operation, enabling shafts to be sunk in difficult ground conditions cheaply and safely. There are three possible methods of constructing a shaft using a bolted segmental lining, they are described below:

i)
Chimney Method

The ground is excavated down to the base of the shaft; the ground may be battered back or supported depending upon the ground conditions. A level concrete base is formed to a slightly larger diameter than the O.D. of the precast rings, (Fig 3). The precast rings are then built from the bottom up to the finished level. The segments are placed individually into position and bolted to the adjacent segments in the shaft until a ring is built. The next ring is built on top of the previous ring in a similar manner. The joints in adjacent rings are usually staggered.

The bottom ring is surrounded by insitu concrete and the ground carefully backfilled around the segments to the finished level.

Fig 2.

Standard Bolted Concrete Segmental Ring Details

DIMENSIONS			Segments per Ring			Volume per Ring	Weight per Ring	Max. piece wt.	Bolts per Ring †				
Int. Dia.	Ext. Dia.	Width		Circle & Cross		Key							
Metres	*Metres*	*Metres*	*O*	*T*	*Key*	*m*^{3}	*Tonnes*	*Tonnes*	*Dia.*	*No.*	*Length*	*No.*	*Length*
1.52	1.77	0.61	3	2	1	0.292	0.70	0.139	¾"	18	6½"	2	10"
1.68	1.93	0.61	3	2	1	0.333	0.80	0.159	¾"	23	6½"	2	10"
1.75	2.01	0.61	3	2	1	0.320	0.77	0.154	¾"	23	6½"	2	10"
1.83	2.08	0.61	3	2	1	0.355	0.85	0.169	¾"	23	6½"	2	10"
1.96	2.21	0.61	3	2	1	0.373	0.90	0.178	¾"	23	6½"	2	10"
2.05	2.33	0.61	3	2	1	0.396	0.96	0.190	¾"	23	6½"	2	10"
2.13	2.43	0.61	3	2	1	0.460	1.11	0.219	¾"	23	7"	2	10½"
2.21	2.51	0.61	3	2	1	0.438	1.05	0.210	¾"	23	7"	2	10½"
2.29	2.59	0.61	3	2	1	0.473	1.14	0.228	¾"	23	7"	2	10½"
2.44	2.74	0.61	3	2	1	0.490	1.20	0.238	⅞"	23	7"	2	10½"
2.59	2.90	0.61	4	2	1	0.532	1.30	0.217	⅞"	28	7"	2	10½"
2.74	3.05	0.61	4	2	1	0.566	1.38	0.230	⅞"	28	7"	2	10½"
2.90	3.20	0.61	4	2	1	0.598	1.46	0.243	⅞"	28	7"	2	10½"
3.05	3.35	0.61	4	2	1	0.622	1.49	0.247	¾"	28	7"	2	10½"
3.20	3.51	0.61	4	2	1	0.654	1.57	0.262	⅞"	28	7"	2	10½"
3.35	3.65	0.61	4	2	1	0.701	1.68	0.279	¾"	34	7"	2	10½"
3.35	3.66	0.61	5	2	1	0.697	1.68	0.240	⅞"	33	7"	2	10½"
3.66	3.96	0.61	4	2	1	0.749	1.80	0.298	¾"	34	7"	2	10½"
3.66	3.96	0.61	5	2	1	0.759	1.86	0.266	⅞"	33	7"	2	10½"
3.96	4.27	0.61	5	2	1	0.847	2.04	0.292	⅞"	40	7"	2	10½"
4.11	4.42	0.61	5	2	1	0.861	2.07	0.296	⅞"	40	7"	2	10½"
4.27	4.57	0.61	6	2	1	0.901	2.19	0.272	⅞"	38	8½"	2	12½"
4.57	4.93	0.61	7	2	1	1.243	2.95	0.328	⅞"	43	8½"	2	12½"
5.28	5.71	0.61	8	2	1	1.569	3.77	0.377	1⅛"	48	9"	2	13"
5.49	5.94	0.61	8	2	1	1.764	4.24	0.424	1⅛"	48	9"	2	13"
5.79	6.27	0.61	9	2	1	1.892	4.55	0.414	1⅛"	53	9"	2	13"
6.10	6.63	0.61	10	2	1	2.436	5.84	0.487	1⅛"	58	10"	2	14"
6.48	7.01	0.61	11	2	1	2.591	6.24	0.481	1⅛"	63	10"	2	14"
6.75	7.32	0.61	11	2	1	2.884	6.92	0.532	1⅛"	68	10"	–	–
7.62	8.28	0.61	13	2	1	3.597	8.64	0.576	1⅛"	73	9"	2	13"
9.25	10.10	0.61	16	2	1	6.032	14.48	0.804	1⅛"	88	11½"	2	15"
10.67	11.53	0.61	18	2	1	6.797	16.26	0.813	1⅛"	98	11½"	2	14½"
11.50	12.11	0.61	18	2	1	5.183	12.21	0.610	1⅛"	98	10"	2	14"
11.90	12.90	0.60	18	2	1	9.234	22.60	1.130	1⅛"	98	12"	2	18"
15.00	15.61	0.61	22	2	1	6.344	15.24	0.635	1⅛"	118	10"	2	14"
18.00	18.86	0.61	30	2	1	10.960	26.39	0.825	1⅛"	158	11½"	2	14½"
19.00	20.00	1.00	22	2	1	22.580	55.20	2.300	M30	118	420	4	210
29.50	30.74	0.67	48	2	1	28.944	68.50	1.370	1⅛"	248	11½	2	15½"
35.00	36.90	1.60	28	2	–	92.030	225.00	7.500	M30	270	575	–	–
38.00	39.22	0.67	62	2	1	36.902	88.65	1.385	1⅛"	318	13"	2	16"

This form of construction is usually only employed to construct very shallow shafts; it is usually easier and cheaper to construct a shaft using one of the following methods:

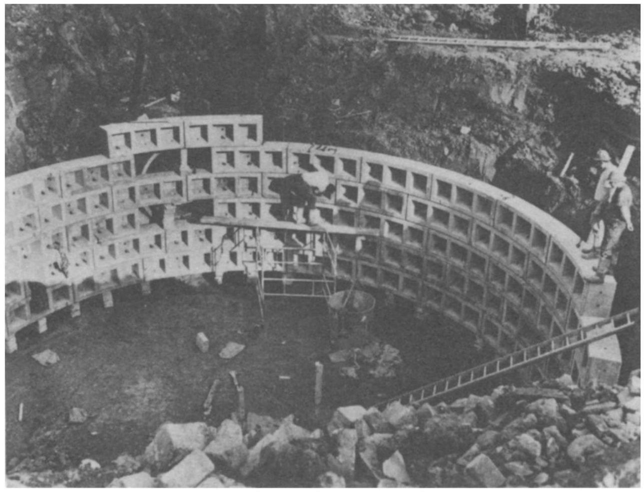

Fig 3. Chimney method.

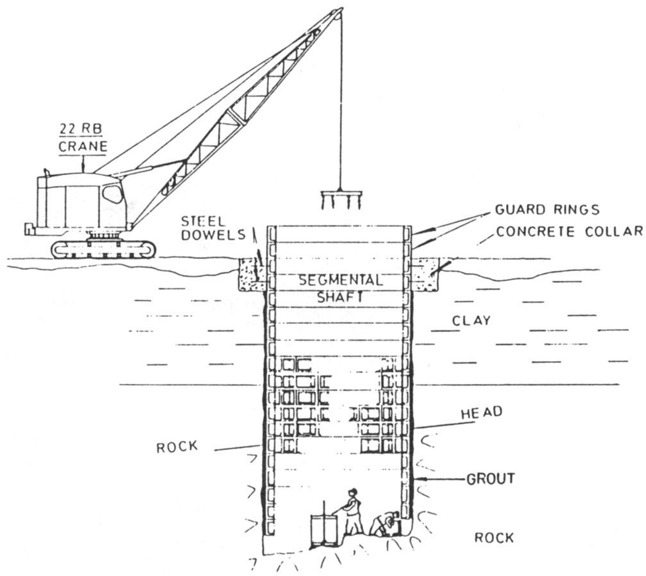

Fig 4. Underpinning technique.

ii)
Underpinning

The top layer of ground is removed to a depth of two or three rings below the proposed level of the top of the finished shaft. A level area is created approximately 500 mm– 1000 mm larger in diameter than the O.D. (Outside Diameter) of the shaft—On a level base the first. two rings are built as described above. The rings are carefully surrounded by insitu concrete to form a rigid concrete collar, (Fig 4). It is important that these two rings be built well; level and to a true circle. The concrete collar serves to preserve the shape and level of these two rings, to protect the edge of the shaft from damage by adjacent construction equipment and to provide a firm “anchor” from which underpinning may proceed.

The ground is then removed in 610mm layers in sections around the bottom of the shaft and segments are bolted beneath the previous ring. When each ring is completed the annulus is filled with grout, the cycle is repeated until the full depth of the shaft is built. If ground conditions allow, more than one ring may be excavated before the annulus is grouted allowing a full shift of excavation and segment erection to take place before grouting.

This form of construction is the easiest and most controlled and will always be preferable when the ground conditions are stable. It allows a shaft to be built accurately and quickly with the minimum of excavation and disturbance to the ground.

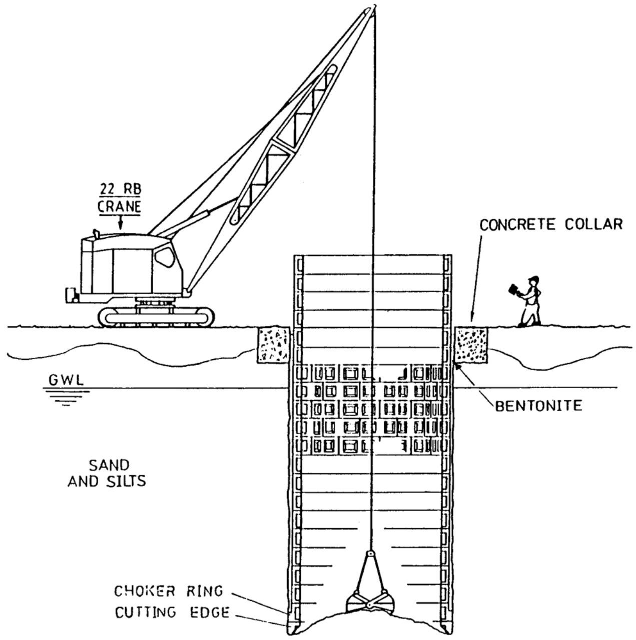

Fig 5. Caisson technique.

iii)

Shaft sinking by Caisson Method.

An insitu collar at least 75 mm larger in radius than the O.D. of the shaft is formed in the ground at the proposed level of the shaft top. Precast segmental linings may be used as the internal shutter by placing key units at each joint to increase the ring diameter and giving the 75 m clearance between the collar and the shaft segments. The shuttering segments are then removed. This collar is usually two rings deep and forms a guide for the sinking of the shaft as well as preventing distortion of the ring and giving protection against damage by construction equipment, (Fig 5).

Segments are built to the correct radius inside the collar with a cutting edge and a choker ring in position. The O.D. of the cutting edge and choker ring is 100mm larger than the O.D. of the standard shaft rings. The 50mm annulus between the lining and the ground is often filled with Bentonite slurry to stabilize the ground and act as a lubricant for sinking the shaft. Segments are added at the top and the shaft sinks under its own weight. Where the self weight is insufficient, kentledge in the form of mass concrete or cast iron weights can be added to help sink the shaft.

(Occasionally hydraulic jacks are used in place of kentledge to give greater control of sinking.) The direction and rate of the sinking is controlled by the correct sequence of excavation from different parts of the shaft and by the position and size of the kentledge weights. The shaft is sunk to the required depth and the base of the shaft is concreted. The annulus between the lining and the ground is grouted from the bottom expelling the bentonite and permanently stabilizing the shaft.

This method of construction has been used successfully for many years and is suitable for unstable ground and in water bearing grounds. The use of Bentonite slurry to sink precast shafts was first used in 1964 at Hammersmith. Its success allowed a much wider use of the caisson method. The accuracy of construction is not as high as the previous two methods and will depend mainly on the ground conditions.

The usually accepted tolerances[5] for shaft construction are:

Finished diameter: 1% or 50 mm which ever is the less.
Vertically over the whole depth: 1:300
Lipping between edges of adjacent segments: 5 mm.

Where the shaft is sunk by caisson the vertical tolerance may be difficult to achieve.

It is quite common to combine the techniques of caisson and underpinning in the construction of one shaft. The shaft is sunk by caisson through unstable ground until stable ground is reached. The shaft is back grouted and the cutting edge removed.

Underpinning then proceeds to the required depth. In both techniques it is usual to stagger the longitudinal joints in adjacent rings; this helps to maintain the shape of the lining and increase its stability.

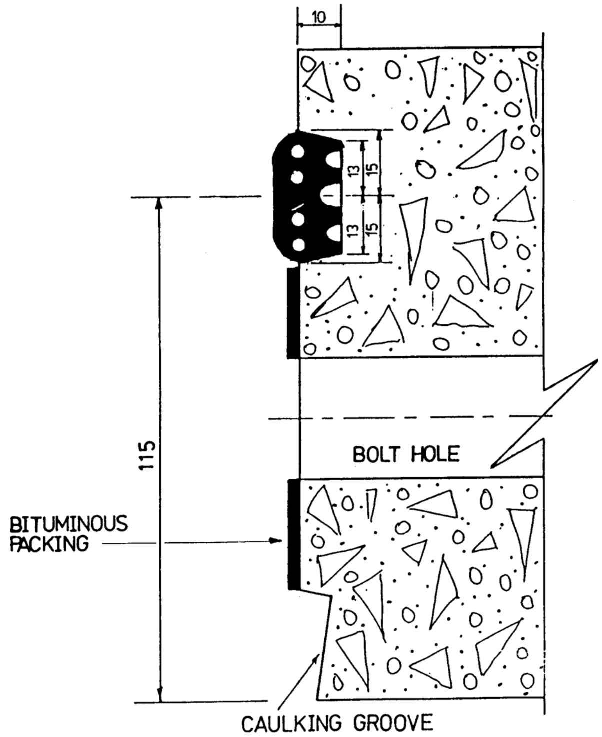

Fig 6. Typical compression gasket joint detail.

Joints are allowed to coincide in adjacent rings near openings in the shaft to facilitate breakouts.

Water Tightness.

The traditional method of achieving water tightness in shafts and in tunnels has been the combined use of back grouting the annulus between the lining and ground and caulking the caulking groove provided on the inner face of all four sides of the segments.

The bolt holes are sealed with the aid of gel grummets under the nuts and bolt heads. About 5 years ago the most widely used caulking material, PC4, which was an asbestos cement mix produced by Expandite Limited, was withdrawn from the market due to the health risks of asbestos. It was eventually superceded with a similar material PC4AF (Asbestos free) but in the mean time two other methods of sealing shaft rings were investigated and met with some success.

Details of these two methods of sealing are as follows:

Rubber Compression Gaskets

An EPDM (Ethylene Propylene Diene Monomer) compression gasket is provided on all four sides of every segment in a ring. The gasket is located in a preformed groove between the bolt hole and the outside diameter of the segment, (Fig 6). Two joint faces of adjoining segments are bolted together and compress the gasket to provide a water tight seal.

The gasket is supplied separately as a complete “window frame” that is preformed prior to positioning on the segment.

The gaskets are carefully stuck into the grooves of the segments on site just prior to use. They form a permanent seal designed to cope with the construction tolerances normally associated with shaft sinking. These gaskets have been used to seal gaps of up to 12 mm against pressures of 2 bar. Pressures approaching 4 bar have been achieved with smaller joint gaps.

Jointing at the corners of the segments is critical and requires special attention. Water Ingress is most likely to occur at a cruxiform joint (i.e. where the corners of 4 segments meet.) and this should be avoided’ if possible. Damage to the concrete around the gasket groove will also impair the efficiency of the seal as will dirt between the seals.

The bolting forces required to achieve full joint closure when a gasket is used are considerably higher than on a non gasketed ring and care must be taken with the segment rib design. However, in many cases standard precast bolted segments have been adapted for use with gaskets. The use of a gasket demands a different method of placing the last segment in a ring and special “top” segments with sloping longitudinal joints have been developed, (Fig 14). The sloping joints allow the insertion of the final segments in the ring without dislodging the gasket. Normally the segment needs to be lowered

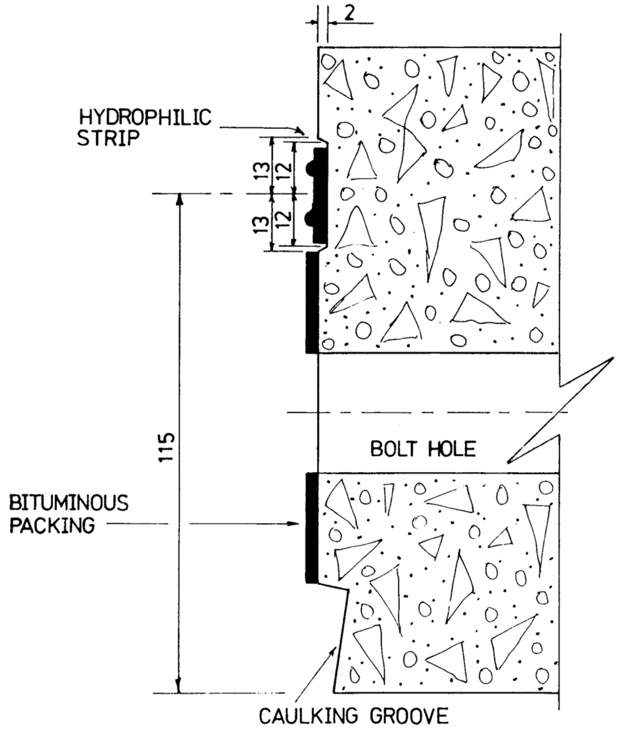

Fig 7. Typical hydrophilic strip joint detail.

approximately 300 mm beneath the ring being built; this can be a disadvantage when underpinning in ground with a short stand up time.

The first gasketed shaft ring was sunk in 1985 at Liverpool, using a 35.5 m. I.D. bolted segmental ring. Since then the system has been used on a large number of different shafts all over the country in both underpinning and caisson methods of shaft sinking. Sizes currently available range from 3.05 m I.D. to 35.5 m I.D.

Hydrophilic Rubber Sealants.

As its name suggests this material absorbs, and swells on contact with water. It is claimed that certain types can expand by upto ten times their volume when left immersed in pure water.

The hydrophilic neoprene rubber strip is positioned on all four sides of every segment in a ring in a similar position to the rubber compression gaskets mentioned above. It is normally recommended that it is located in a small groove approximately 22 mm wide and 2 to 3 mm deep, depending upon the size of the strip, (Fig 7). Adjoining segments are bolted together and if water is present the material expands to provide a seal.

The material is supplied as a continuous strip which is cut off to length to suit the segment size. The strip is glued in the preformed groove starting at one corner of the segment and positioned along all four sides, returning to the start point.

The hydrophilic qualities of the rubber mean that it must be kept dry prior to use in the shaft, if it is applied to segments before they are needed they must be kept under cover and dry.

The material is much thinner than the rubber compression gasket and is thus less intrusive. It may therefore be used with standard bolted segments. The full expansion of the rubber may take several days to occur and requires full immersion; therefore it functions best where there is a large amount of water present. The intermittent presence of water does not allow it to develop its full capacity and leakage may result. To be fully effective it should be compressed to provide an initial seal. Care must be taken when cutting and fixing the strip on site.

The system has been used on many contracts throughout the U.K. and can be provided on most sizes of standard shaft ring from the manufacturers. It is currently in use on the 10.30 m I.D. smoothbore shafts on the London Ring Main mentioned later in this paper.

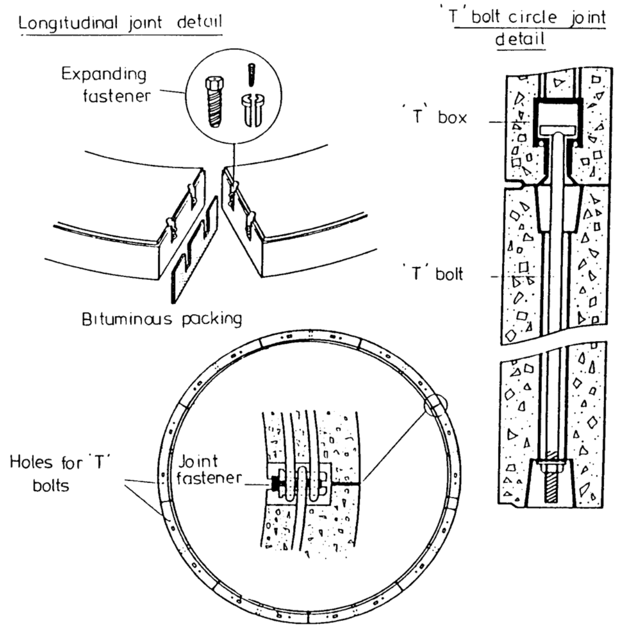

Fig 8. Charcon "One pass" shaft lining.

Smoothbore Shafts.

In some cases the finished shaft is required to have a smooth inner surface and the traditional way of achieving this is to provide an insitu concrete lining. Alternatively infill pellets[3] may be provided to fill the recesses of the standard bolted lining. When a smoothbore is required large economies of time and money could be saved if this could be provided on the precast lining thus allowing one operation to give the finished shaft. The problem for the precast manufacturers is to produce a precast segmental ring that has a strong mechanical connection between segments and between rings. The connection must be suitable for both the underpinning method of operation, where the lining has to support rings beneath it before grouting, and also the caisson method of shaft sinking where the lining is subjected to dynamic loading during the downward movement of the shaft. The fixing of course must be strong and easy to place but not interfere with the smooth inner surface of the lining. Several different systems have been developed to provide a nearly smoothbore shaft which only requires caulking to provide a smooth finish.

The two smoothbore shaft linings currently available are the "Onepass" shaft lining manufactured by Charcon Tunnels Limited and the "Bucline" shaft lining manufactured by C.V.Buchan (Concrete) Limited. The onepass[4] lining incorporates a special fixing at the cross joints, (Fig 8) and T bolts for the circle joint fixings. The Bucline system[3] (Fig 9) consists of a turnbuckle and steel dowels. These shafts are offered in sizes between 2.44 m and 10.3 m I.D. and they have been used successfully in a number of shafts throughout the country.

The smoothbore precast linings are more expensive than a standard bolted lining of the same diameter but if a smooth internal finish is required the total shaft cost will be less. A disadvantage to these smoothbored shafts is that, in the absence of an insitu lining, construction mistakes cannot be easily corrected. This is often an important factor when the shaft is sunk by the caisson technique.

Impressive examples of these smoothbore shafts have been constructed recently in London for the Thames Water Authority during the construction of the London Ring Main. The deepest shaft is at Barrowhill and has a 10.30 m I.D. smoothbore lining to a depth of 75.0 m below ground level in London clay. (Fig 10). The lining has been designed to cater for lateral pressures up to 1½ times the overburden pressure in the heavily over consolidated clay.

The solid precast rings vary in wall thickness from 300 mm up to a maximum of 720 mm and were manufactured with a 50 N/mm^2 concrete. The rings were 1.00m deep. Each ring consists of 14 ordinary segments, two top segments and a solid parallel sided key. The heaviest segments weighed 3.7 tonnes and the maximum ring weight was 60 tonnes. The total weight of the precast concrete in this shaft was approximately 2800 tonnes.

The upper section of the shaft was sunk by caisson technique using 11.90 m I.D. standard bolted rings through water bearing soil and then changed over to the smoothbore rings and underpinning when safely in the London clay, (Fig 11). As mentioned previously, a hydrophilic sealing strip was used on these

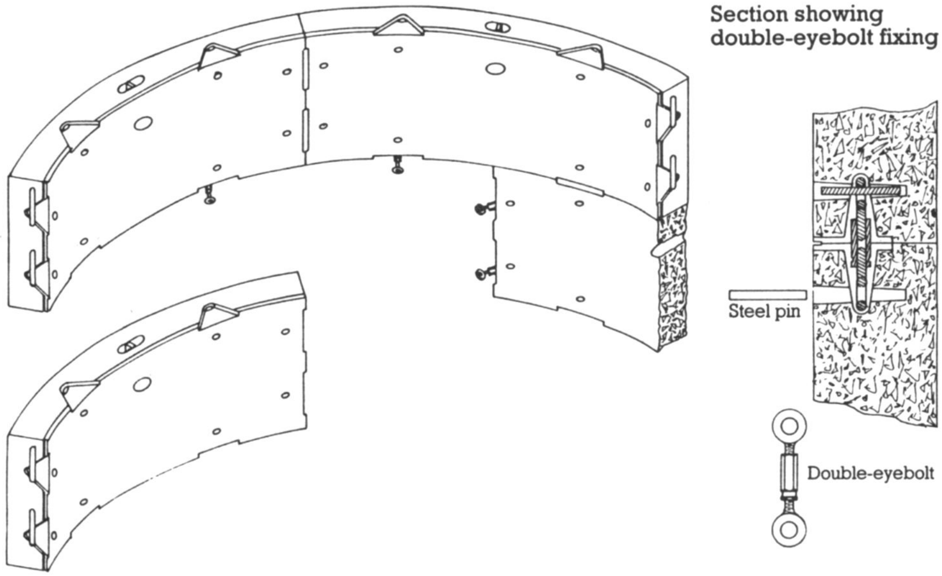

Fig 9. C.V. Buchan "Bucline" shaft lining.

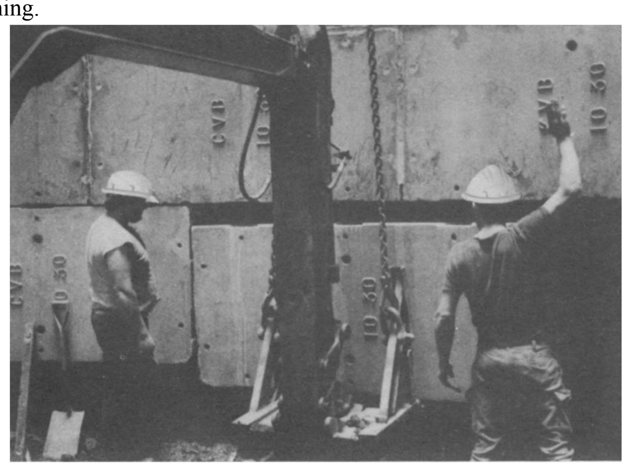

Fig 11. Underpinning a smoothbore segment in London clay.

Large Diameter Bolted Shafts

Several large diameter shafts have been sunk within the last five years using bolted segmental precast concrete rings. Details of two of the largest, which were designed as pumping stations, are given below.

A 19.0 m shaft for the Northumbrian Water Authority, at Seaton Valley in Tyne and Wear. These rings were underpinned through a hard mudstone to a depth of 10.00 m. The ground was excavated by drilling and blasting to loosen the base and the ground then trimmed and excavated with a hydraulic excavator with a jigger attachment, Fig 12.

Each ring consisted of twenty two identical ordinary segments, two slightly smaller "top" segments and a small solid key. The segments were 500 mm thick, 1000mm deep, 2500 mm long and each weighed 2.3 tonnes. The total ring weight was 55.2 tonnes.

The largest precast shaft sunk in recent years was the 35.5 m I.D. bolted segmental lining on the MEPAS scheme (Mersey Estuary Pollution Alleviation Scheme) on the Liverpool docks, (Fig 13). The precast rings are thought to be the largest ever cast with a complete ring weight of 225 tonnes each consisting of thirty segments, 700mm thick, 1500 mm deep 3800mm long and weighing 7.5 tonnes, (Fig 14).

The shaft was sunk to a depth of 20m through clay into weathered sandstone adjacent to Sandon dock and because of the size of the shaft the design included checks on the overall stability of the lining and on localised buckling. The lining was designed to accommodate a maximum allowable ground load of 240 KN/m^2. The ground water level was approximately 2.5 m below ground level and the ground was dewatered during construction using seven 40m deep wells.

The ring employed a large E.P.D.M. rubber gasket seated in a preformed groove in the segments located behind the bolt holes.

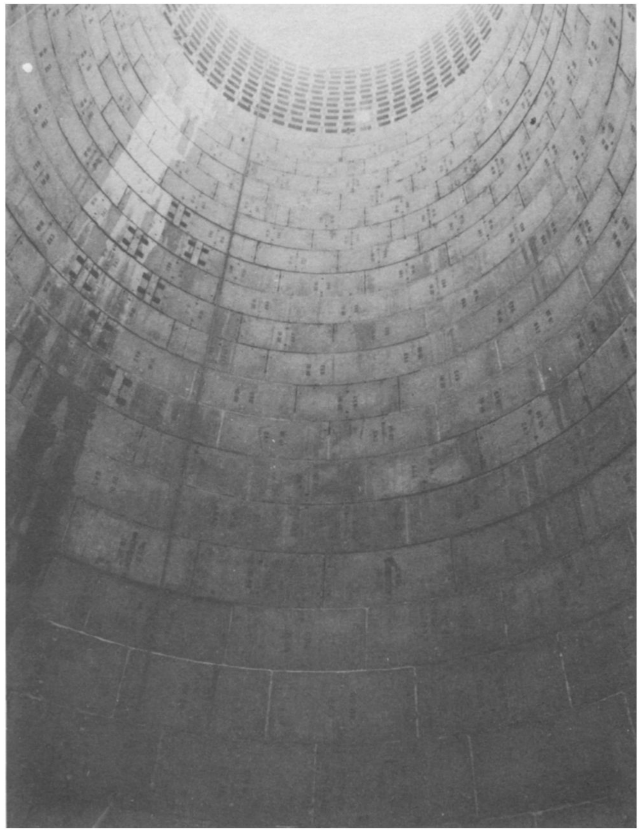

Fig 10. Barrowhill shaft.

The shaft, was sunk by underpinning from the surface and the segments were hung using a specially designed counter balance frame.

Storage Bunkers and Spiral Chute Bunkers.

Smoothbore storage bunkers have been specifically developed for the storage of bulk materials such as coal at great depths in British Coal Mines. Typically the shafts are between 6.0 m and 7.5 m I.D. with a solid wall thickness of 225 mm. The 1.0 m deep rings have between twelve and fifteen segments, each weighing 0.9 tonne. The bunker shafts are capable of withstanding rock pressures of greater than 1000 KN per m^2.

The optimum finished internal diameter of the Bunker probably lies between 6 m and 7.5 m and the diameter of the excavation between 7 m to 9 m, for smaller diameters the cost becomes high per ton of material contained, while larger diameters present problems of construction and support in the mine. Structurally the height of the bunker is not a design limitation but it is usually less than 100 m. Where the bunker height is greater than 10m the stored material should not be allowed to fall freely and some form of chute is necessary to prevent damage and excessive wear to the base and side walls as well as to reduce degredation and the risk of incendive sparking.

To overcome this problem the use of a peripheral spiral within the bunker wall has been developed, (Fig 15). A bolt on peripheral spiral chute was also developed at Calverton Colliery, this bolted on to the inside of the shaft wall. On the spiral bunkers a helix angle for the chute has been adopted to provide a terminal velocity in the order of 7 m to 8 m per second in order to minimise wear and degradation and yet still allow the spiral to be self cleaning when the bunker empties. The velocity must also be such as to ensure that the material clings to the periphery of the spiral.

These units have been installed in many British Collierys at depths approaching 1000m below ground level. The most recent being at Stillingfleet, Riccall and Silverwood Collierys. The units are simple and fast to build and are joined using the same smoothbore fixings described earlier in this paper. The bunkers can also incorporate man access shafts at the rear of the main shaft, and individual segments can easily be modified to incorporate local structures, vent units or monitoring devices for capacity.

Fig 12. 19.0 m I.D. precast shaft rings at Seaton Valley, Tyne & Wear.

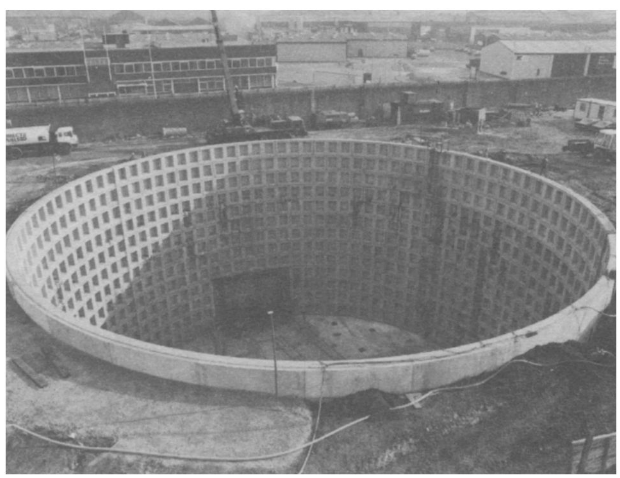

Fig 13. 35.5 m I.D. precast shaft at Sandon Dock, Liverpool.

Ancillary Items

Over the years a number of ancillary items that could be readily precast and hence speed up the construction times have been developed. Amongst the first to be developed were the choker ring and concrete cutting edge for use when caisson sinking. These are now available for most standard sizes of shaft, (Fig 16). The choker ring is made up of standard segments 50 mm thicker over half their depth. They "choke" off the annulus (Fig 5) between the lining and the ground and prevent ground loss when the cutting edge is removed as well as providing a bentonite seal at the bottom of the shaft during construction.

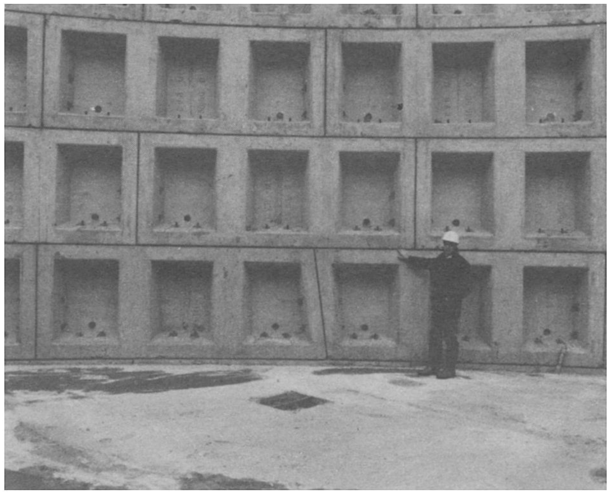

Fig 14. View of segments at the bottom of Sandon Dock shaft.

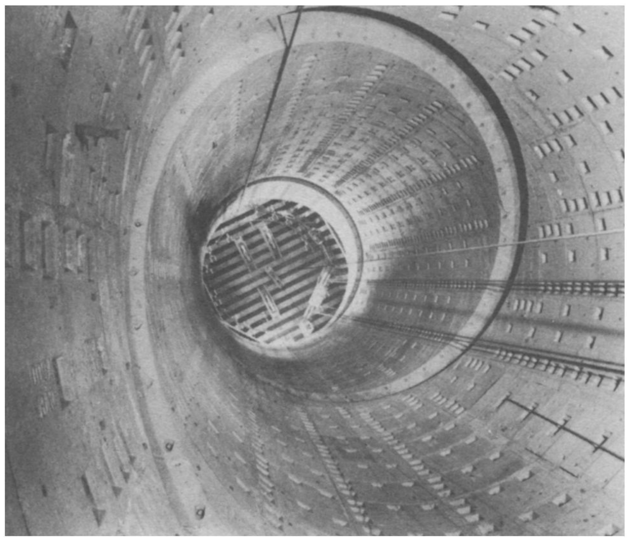

Fig 15. Spiral chute bunker.

The cutting edge trims the sides of the excavation and helps provide greater control during sinking.

Precast Reinforced Concrete Cover Slabs are available for shaft diameters 1.52m upto 7.62 m I.D., the larger sizes are made up of match cast units laid side by side, (Fig 17). They can be designed to suit most arrangements of opening and are normally designed to accept 45 units of HB wheel loading to BS5400 part 2, together with ground loading at depths between . 75 m to 4 m below ground. Cover slabs for larger diameter shafts have been provided up to 15 m I.D., these units are usually prestressed and it has been found more economical to provide a composite construction in the form of prestressed inverted “T” units with a structural concrete infill. Intermediate landing slabs within the depth of the shaft can also be provided for most diameters of shaft.

Platform support rings and corbel rings have been developed to support slabs within the shaft. A Platform Support Ring is a separate ring of concrete placed between two rings in a shaft providing a reinforced concrete nib to support the landing slab. The corbel ring is a similar support but has been provided within the depth of a standard ring thus simplifying the design of the shaft and reducing the construction time. Platform support rings and corbel rings are normally designed to accommodate a landing slab live loading of 10 KN per m^2 across the whole

Future Trends

Increasing use of off site fabrication has been an observable trend throughout the construction industry; a movement towards factory finished products and fast construction.

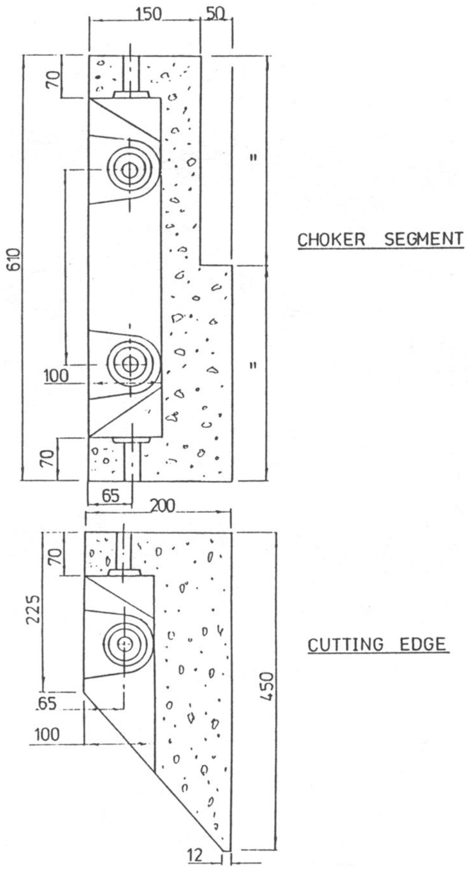

Fig 16. Typical section through a cutting edge and choker segment.

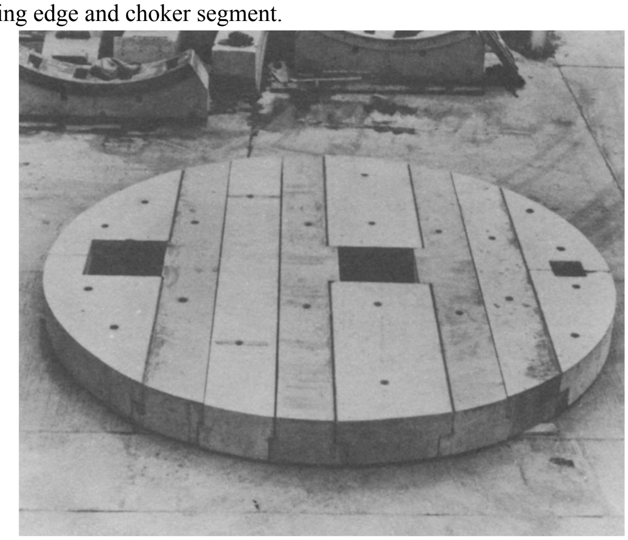

Fig 17. Cover slab to suit a 7.62m I.D. Shaft.

This trend is evident in shaft construction; it shows itself in the search for the ideal shaft lining: a smoothbored and water tight lining, produced under factory conditions off site, ready to be built as quickly as the ground can be removed, requiring no finishing works, suitable for all ground conditions and cheap to produce.

Unfortunately such a lining does not exist, nor is it likely to, but we can expect much greater use of both the smoothbore shaft linings and the proprietary sealing systems.

The current sealing systems are not perfect; they require careful supervision during construction if leaks are to be avoided. If a leak does occur it is difficult to isolate and it may be necessary to caulk the whole shaft to seal it. We can expect the industry to improve these systems and overcome the drawbacks.

Where ground conditions permit we can anticipate a trend towards larger segments; in height and in length, in an attempt to reduce handling and erection costs. The number of joints will be reduced and so therefore will the cost of rendering the shaft watertight.

Despite these future trends, it must be remembered that the traditional method of lining and sinking a shaft using ribbed bolted concrete segmental linings is cheap and safe and there are a large range of different diameters readily available from the manufacturers. Whether sunk by underpinning or by caisson techniques, whether the shaft is lined insitu or not, in most ground conditions the traditional lining has proved its worth. The ring has an efficient section for resisting thrusts and bending moments and it is easy to build and is tolerant of error. This traditional method of construction will be hard to replace.

References:

1. WATSON, D.M. West Middlesex Main Drainage. Jnl.Instn.Civ.Engrs.1937, pp463–617.
2. GROVES, G.L. Tunnel Linings with special reference to a new form of reinforced concrete lining. J.Instn.Civ.Engrs.1943, 20, March No.5 pp29–64.
3. C.V.BUCHAN LTD—Tunnel & Shaft Brochures.
4. CHARCON TUNNELS LTD—Tunnel & Shaft Brochures.
5. WATER AUTHORITIES ASSOCIATION: Civil Engineering specification for the Water Industry (Second edition) publ. Water Research Centre.
6. TUNNELLING ACCESSORIES LTD—“Internal report on tests on elastomeric sealing gaskets”.
7. VICTAULIC INDUSTRIAL POLYMERS —“Tunnel segment gasket and jacking pipe seal brochure”.

Current development in conventional and mechanical shaft sinking in the Federal Republic of Germany

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In the course of the paper the development in both conventional and mechanical shaft sinking methods in Germany over the past 20 years is illustrated. The main point of emphasis, however, will be a description of the modern mechanical shaft sinking method, a subject which could not have been dealt with 20 years ago as this technique did not exist at that time.

Conventional sinking techniques have benefitted in particular from improved design of winders and the application of multi-deck stages.

In the past when sinking large shafts, two winders were normally used which doubled costs for foundations, installation, depreciation and dismantling. Modern designs use only one hoisting machine, be it a bobbin winder or a double drum machine. Modern layouts make it possible for this type of equipment to undertake the function of two winders, achieved by a special layout of motors and gearboxes in order that the machine can be operated by one winder driver or, in its declutched state, using the drums or bobbins as single units by two operators. In Germany, bobbins are still used predominantly but rope problems are experienced when hoisting large loads from great depth which, in the author's opinion indicates the end of this technique with the consequence that drum winders will take over. Normally one deep sinking requires at least two or three flat rope sets, whilst double drum winders can use one rope set for several projects, therefore being more economical in that respect.

It has been proved e.g. when sinking the two new shafts at Ensdorf mine from 1981 to 1985 that these winders are capable of achieving sinking rates of 80–100 m per month to a depth of 1324 m in a shaft of 7,5 m internal diameter. Such performances are clearly marker points when consideration is given to shaft sinking by blind boring which, for the time being, cannot compete with optimal operated conventional sinking systems. This, however, is the author's personal opinion.

Conventional Equipment

In multi-deck stages with concrete as the widely preferred lining element, the stage hoist ropes are used to guide the kibbles with the cactus grab suspended below the bottom deck.

For the shaft furnishing operations a different multi-deck stage system is used and rates of 60 metres per day have consistently been achieved, even in shafts with complicated furniture (Hünxe shaft; 1982–1987).

In Germany, compressed air powered drill jumbos are used as it is generally felt that hydraulic powered jumbos are too sophisticated for this purpose in a shaft. Recent experiences also show that contrary to previous assumptions it is not the size of the kibble which is the determining factor for the achievable sinking rates but the capacity of the mucking system. This was proven exactly whilst sinking the 8.0 m diameter North Shaft Ensdorf (1981–1985) of 1000 m depth.

OBSERVATION AND CONTINUOUS MEASURING OF THE ICEWALL THICKNESS IN FROZEN SHAFTS

100 years ago freezing of unstable and water bearing ground was introduced to shaft sinking. It relies on the diligent collection and evaluation of a multitude of geotechnical data which is entered into formulae proved and refined over time. Shafts flooded as a result of leaks in the icewall, bear witness to the remaining uncertainty caused by unforeseen changes in homogeneity and isotropy.

Since the mid 1950's the theoretical calculations have been tested and confirmed by using ultrasonic techniques to measure the thickness of the icewall at different horizons. As a rule, this is only a one off check for each shaft with no continuous monitoring of the state of the icewall, since this process requires the interruption of sinking operations and the use of expensive and skilled specialists.

A method recently developed by Thyssen, however, provides for a continuous monitoring os the icewall. It is based on a computer simulation which determines the icewall thickness at a predetermined horizon using data such as differential brine and rock temperatures at different depths, accounting for freeze hole deviation, surface ground temperatures and others. These data are processed and the icewall thickness at the chosen horizon can be determined by using graphical means.

By repetition of this process a number of horizons can be investigated and by computing the results an outline of the inner and outer icewall can be traced by a plotter.

Obviously this system relies on a considerable number of sensor points at various locations in each horizon but promises cost advantages in terms of better decision making regarding the start of sinking and anticipating and preventing possible problems.

This was evident in the Hünxe (West Germany) shaft in October 1983 when the total freeze plant had to be switched off for a short period of time. Fears of losing control through a possible thaw were allayed by using this method and it was found that no significant change in the icewall had occured.

SHAFT LINING (COMPOUND STRUCTURE) IN FROZEN SHAFTS

For frozen shafts, two lining systems are available, the (sandwich) compound structure and various types of in-situ linings. The choice of application depends mainly on the anticipated stress to which the shaft lining will be exposed, especially due to the extraction of coal seams. It is commonly held that if the bend radius of the shaft lining is limited up to 3000– 5000 m the decision would be in favour of the compound structure. In-situ linings can consist of mass or reinforced concrete, tubbings or brickwork. To obtain water-tightness the in-situ lining can be combined with plastic sheating consisting of 2–3 mm thick sheets welded togehter to form a con tinuous vertical sleeve.

The compound structure consists of the preliminary lining, a 100–300 mm asphalt filled joint behind a 8–12 mm thick steel cylinder and the structural inner lining of between 0.5 to 1.2 m thick (reinforced) concrete which can be further strengthened by an inner steel cylinder, the purpose of which is to render additional structural strength at greater depth.

The purpose of the preliminary lining is to protect the sinking crew against falling rocks, but it may be designed in some cases to add stability to the frozen ground.

In general, concrete blocks are layered, the vertical and horizontal joints are not mortared, but filled with plywood and backfilled with mortar to form intimate contact with the strata.

On continuation of sinking, ground convergence causes these loosely packed rings to go into compression taking up the early resistance of the packing and thereafter relying on the compressive strength of the concrete blocks.

Problems have occurred when this type of lining was exposed to out-of-balance ring pressures and remedial work had to be done, but generally this type of preliminary lining has proven to be useful and reliable with up to 10% out-of-balance ring pressure. If, however, the out-of-balance pressure of the rock exceeds 10% of the isotropic strength, other preliminary lining systems have to be used. This was the case in 1989 in the Gorleben shaft when high quality (very fine-grained) steel rings were used in order to resist an out-of-balance rock pressure which—in a 15 m clay layer—was 50% higher than the homogeneous rock strength.

MECHANICAL SHAFT DRILLING

This method is of increasing significance in countries with high labour costs.

Firstly however, one must distinguish between the rod-guided and the rod-free shaft drilling methods.

a) Most of the rod-guided methods have the disadvantage that it is difficult to control their verticality because of the lack of adequate technical facilities. Sinking performances and economy depend primarily on the compressive strength of the rock and its changes. There is also a limitation in depth in all cases where the shaft lining is floated into position. Therefore, the application of this procedure is limited to particular cases.
b) The rod-free method can be operated either with or without a pilot hole

Worldwide experiences prove that there is to date no blind shaft boring machine ie. without a pilot hole, which operates really successfully. The problem is to remove the drill fines from the bottom of the shaft as quickly as they are created. For this purpose mechanical, hydraulic and vaccum systems have been used but none has yet fulfilled expectations.

On the other hand, shaft boring machines operating on a pilot hole have achieved drilling performances which, until now, are unsurpassed. In this system the drill fines are disposed via a pilot hole to the bottom of the shaft.

A pre-requisite of this method, however, is that there is already access to the bottom of the shaft to be drilled.

Rod-free shaft boring machines, however, are completely steerable, even in cases where the pilot hole deviates from the shaft axis.

All lining systems nowadays can usually be used with this technique. In addition, shafts through very hard rock can be unlined as already practiced in the past. In all other cases rock bolts with wire mesh, steel rings or concrete can be placed, directly above the machine.

When using concrete as a permanent lining material, the sinking performance is not determined by the drilling performance of the machine but by the setting rate of the concrete. For example, the Lummerschied shaft of Saarbergwerke was sunk in 1989 with a daily performance rate of 8 m down to 1000 m depth. Such modern shaft boring machines can drill from 6.00 m to 8.50 m diameter.

As far as the author is aware the best drilling performance rate up to now was achieved in Alabama (USA) in 1982 in No. 4 North Shaft of Jim Walter Resources by drilling a 7.0 m diameter shaft of 575 m depth within 32 days with a best recorded performance of 37 m in a single day.

An interesting alternative to shaft drilling is, of course “raise-boring”. This applies particularly to depths of up to 400 m with shaft diameters of 5 m provided the rock does not exceed a certain compressive strength. Generally it can be stated that the raise boring method applied to shaft diameters of such dimensions is more economical than the shaft drilling method. However, as with all other rod-guided methods it has the disadvantage that it is not possible to achieve absolute verticality, a requirement for hoisting shafts. It is eminently suitable where verticality is of secondary significance e.g. ventilation shafts and ore chutes. The geological conditions, however, have to be ideal, i.e. that in the case of tectonically disturbed rocks large blocks of rock can become displaced and lead to blockages of the cutter head and with resultant costly string breaks, possibly seriously damaging or even destroying the cutter head.

Of particular interest is the application of the drilling method to the sinking of frozen shafts.

This method was applied for the first time in 1985 in Canada by drilling a frozen shaft (of 21 m frozen depth). The principle was to freeze the shaft core and the shaft walls completely. After that, the pilot borehole was drilled in the middle of the frozen shaft and the shaft was reamed to its final diameter of 8 m by application of the raisebore method. This shaft was drilled within 3 days and lined with in-situ placed concrete within 8 days.

Finally let me make some remarks about drill string operated shaft drilling. The principle of this method is as follows: A screwed or flanged drill string carries a cutter head with a drill collar of nearly the same diameter positioned above it. Normally, it is rotated by means of a turntable. Suspended by a multi-fall rope purchase, this system is carried by a regular drill rig with a high hook load capacity. The disposal of the drill fines is accomplished through the drill string using a pump or by mean; of the air-lift principle. The drill mud of a density of >1 circulating in the drilled part of the shaft serves to support the shaft walls.

This method is not as yet perfectly steerable. The disposal of the drill fines can create difficulties in cases where clays have to be drilled as they usually block the cutter head completely. Also in rocks of high compressive strengths, the bit wear is enormously high.

In solid, non-water-bearing rocks, 4.0 m diameter shafts of 700 m depth have already been drilled by this system, e.g. in 1985 Betews shaft in Wales.

For the purposes of American underground atomic tests from 1960–1980 several hundred shafts of up to 1.80 m diameter were drilled to a maximum depth of 1850 m.

Shafts in water-bearing ground require a waterproof lining which has to be floated into the shaft.

Such a technique, however, is limited by depth because of the increasing wall thickness of the support. Normally a “sandwich” lining is used consisting of a double-steel cylinder, the annular space of which is filled with concrete. The cylinders which are about 4 m high are welded together at the surface and floated into position.

To the best of the author’s knowledge this method is limited by a diameter of 4 m and a depth of 400 m but is particularly suitable for the drilling of small diameter ventilation shafts.

The most recent development in this field, however, is the availability of a steerable “down the hole” cutter head manufactured by Wirth, Germany, which has already been successfully used in 1988 in West Germany and Russia. This technique is expected to revolutionise the drill string operated shaft boring system in the future.

SUMMARY

Over recent decades, conventional shaft. sinking techniques have developed from pure manual tasks of blasting, mucking and lining, to the mechanization of these operations and has, as a result, achieved technical standards which, in the author’s opinion, will be difficult to improve in the near future.

In particular, the extremely high increase of labour-costs over the past 15 years has led to the development of the shaft boring technique which has been the decisive turning-point in cost reduction for shaft sinking.

Because of the current and the expected market situation there is little chance of utilising modern shaft boring techniques in Germany, but there is legitimate hope that shaft boring will experience a widespread application in other countries, especially in those which are, as a result of their also increasing labour-costs, obliged to apply more cost-advantageous methods to the development and exploitation of their mineral resources.