Q. ZÁRUBA, V. MENCL

LANDSLIDES AND THEIR CONTROL

DEVELOPMENTS IN GEOTECHNICAL ENGINEERING VOL 31

COMPLETELY REVISED EDITION

ISBN 0-444-99700-8 (Vol. 31)

LANDSLIDES AND THEIR CONTROL

Second completely revised edition

QUIDO ZÁRUBA, VOJTĚCH MENCL

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The first edition of the book, published in 1969, was accepted favourably, as is evidenced by the fact that it was soon out of print and was translated into Japanese and Romanian. This second edition maintains the overall conception of the first: it presents an overview of theoretical and practical aspects of slope movements and of the methods of their investigation, prevention and control. Numerous case histories, many of them from the authors' own experience, illustrate the various facets of the landslide problem.

The theoretical chapters discuss the mechanism of natural and man-made landslides in relation to the state of stress in rocks, to the mobilization of their shear strength, uplift, groundwater conditions and the chemistry of weathering processes.

For practical purposes, new methods, concerning both the investigation of threatened slopes and modern control measures, are described in some detail. With respect to the growing relevance of landslide problems due to the limited choice of appropriate building sites and the increased boldness of structures, which have in turn necessitated a more intense study of slope movements, all chapters have had to be updated and complemented by new knowledge. It was particularly Chapter 4 (Mechanics of the development of slope failure), Chapter 6 (Methods of landslide investigation) and Chapter 7 (Stability analyses) which required a new approach, as the introduction of new methods of investigation and the applica-

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Developments in Geotechnical Engineering 31

LANDSLIDES AND THEIR CONTROL

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by

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and

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ELSEVIER SCIENTIFIC PUBLISHING COMPANY

Amsterdam · Oxford · New York · 1982

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Translation Editor
Mark Cowan

Published in co-edition with
ACADEMIA, Publishing House of the Czechoslovak Academy of Sciences, Prague

Distribution of this book is being handled by the following publishers

for the U.S.A. and Canada
Elsevier Science Publishing Co., Inc.
52 Vanderbilt Avenue
New York, New York 10017

for the East European Countries, China, Northern Korea, Cuba, Vietnam and Mongolia
Academia, Publishing House of the Czechoslovak Academy of Sciences, Prague

for all remaining areas
Elsevier Scientific Publishing Company
1 Molenwerf
P. O. Box 211, 1000 AE Amsterdam, The Netherlands

Library of Congress Cataloging in Publication Data

Záruba, Quido.
Landslides and their control.

(Developments in geotechnical engineering; 31)
Translated from the Czech by Mrs. H. Zárubová.
Bibliography: p.
Includes index.
1. Landslides. 2. Slopes (Soil mechanics)
I. Mencl, Vojtěch, engineer. II. Title. III. Series.
QE599.A2Z3713 1982 624.1'5 81-19541
ISBN 0-444-99700-8 (Vol. 31) AACR2
ISBN 0-444-41622-5 (Series)

Printed in Czechoslovakia

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PREFACE

The first edition of the book, published in 1969, has been out of print in both its Czech and English versions for several years. At the invitation of Elsevier, we have prepared a new, second edition which, considering the rapidly growing interest in the problems of landslides and their study within the last ten years, has had to be revised and very considerably expanded.

During the latter period there have been many conferences on this theme: in 1969 there was the Seminar on Geological Engineering Aspects of Landslides, in Edmonton, in 1971 the Conference on Natural Slope Stability, in Naples, and in 1972 the Symposium on Landslide Control, in Japan. Landslides were one of the subjects covered in the Cannes Symposium of 1973, at a further meeting in the Soviet Union in 1975, and at the 1977 Symposium in Prague; the problems of the stability of slopes now appear on the agenda at all international geological congresses. A survey of the proceedings and reports of these meetings shows that much valuable knowledge has been accumulated, but also that many problems still remain to be solved. The introduction of new methods of investigation and the application of computing techniques has brought much progress with respect to theoretical analyses and data handling.

It has thus not been easy to select the best material from the wealth of new information, since we did not wish to expand the new edition too much, preferring to maintain its character as a compact handbook.

We have omitted from the second edition the chapter on the occurrence of landslides in Czechoslovakia which was of interest mainly to Czech readers; on the other hand, we have included four new chapters, one on landslides in dam construction (10), one on road construction (11), one on urban planning (12) and one in the exploitation of mineral deposits (13). These chapters include new examples arising from our own practical experience.

The chapters which deal predominantly with the geological aspects of the subject (1, 2, 3, 5, 6.1–6.17, 10 and 12) have been prepared by Q. Záruba and translated by Mrs. H. Zárubová while the remaining chapters which discuss the subject from the point of view of soil and rock mechanics have been written and translated by V. Mencl.

Acknowledgements: The authors extend their thanks to Doc. Ing. J. Škopek, CSc. and Ing. J. Rybář, CSc., who edited and revised this book, for their expert and valuable comments. They are also indebted to Elsevier Scientific Publishing Company for a careful revision of the English translation of the text.

SYMBOLS

a	radial distance
B	shear strain at which dilatancy arises
c'	shear strength under zero normal stress in terms of effective normal stresses
D	depth; the relative increase in thickness of the shear surface owing to dilatancy
E	normal force acting between two slices in stability analysis; deformation modulus in general
E_d	dynamic deformation modulus found by geophysical exploration
E_m	immediate deformation modulus
F	factor of safety
FEM	finite element method of mechanical analyses
h	height in general; piezometer height
I_1	first invariant of stress, $= \sigma_1 + \sigma_2 + \sigma_3$
I_{1d}	I_1 at which dilatancy turns into contractancy
K	coefficient of lateral stress (ratio of the magnitudes of horizontal and vertical stress); force in rock bolt; Mohr's circle
K_0	natural (intrinsic) K
Δl	horizontal displacement; length of a section of slip surface
p	potential shear surface
P	force in general
q	unit load on a foundation or on a field test plate
q_f	bearing capacity
r	radius
Δr	increase in radius
R	resultant; Skempton's correction factor in stability analysis
s	developed shear surface;
S	earth pressure (a force); resultant shear resistance
S_0	earth pressure at rest
S_r	degree of saturation
T	tangential (or shear) force
u	horizontal displacement; ordinate of the loading diagram of uplift

U	uplift force
w	water content; vertical displacement
w_L	liquid limit
w_P	plastic limit
W	weight
X	vertical interslice tangential force
α	angle of inclination of the slip surface; angle between the bolt axis and the normal to the slip surface
β	slope angle, dip angle
γ	unit weight
γ_w	unit weight of water
ν	Poisson's ratio
σ	normal stress
$\Delta\sigma$	increase in stress
σ'	effective normal stress
$\sigma_1, \sigma_2, \sigma_3$	major, intermediate, minor principal stress
σ_h	horizontal normal stress
σ_v	vertical normal stress
$\sigma_x, \sigma_y, \sigma_z$	normal stress acting on planes normal to X, Y, Z axes
τ	tangential stress; shear resistance in general
τ_f	shear strength
τ_r	residual shear resistance
φ	angle of shear strength
φ'	angle of shear strength in terms of effective normal stresses
φ_r	angle of residual shear resistance
ϕ	equipotential line of a flow net
ψ	flow line of a flow net

Chapter 1

INTRODUCTION

1.1 Definition and study of mass movements

The problem of the stability of slopes, both natural and excavated, has to be faced in many fields of human activity, particularly in civil engineering. When slope stability is disturbed, a great variety of sliding movements takes place.

Rapid movements of sliding rocks, separated from the underlying stationary part of the slope by a definite plane of separation, are designated as landslides in the strict sense.

Sliding phenomena also include slow, long-term deformations of slopes, which usually occur whithin a thick zone involving a system of slide surfaces. These deformations possess the characteristics of a viscous movement and are referred to as "creep".

On steep rocky walls, blocks of solid rocks may become loosened and fall to the foot of the slope. Such a rapid movement is termed a rockfall.

Our book does not deal with avalanches, a term we think should be restricted to mass movements of snow, in agreement with Rapp (1960). Subsidence, i. e. the vertical settling or sinking of the earth's surface, is also beyond the scope of this work.

Landslides and other slope movements have attracted the attention of man in the same way as other uncontrollable natural phenomena (earthquakes, volcanism, floods) which threaten his life or property. In some regions, landslides occur rarely, whereas in others they are so frequent that they represent an important factor in the modelling of the landscape forms. Because of the great damage they cause to farmland, forest stands, communications, engineering structures and buildings, they are also a serious economic problem.

The sliding of slopes is not uncommonly caused by human activity, such as deforestation, bad construction methods, etc. The great diversity of forms and the complexity of interrelationships, as well as the practical relevance of landslides, can be recognized only by systematic and thorough study.

Landslide phenomena are usually studied from two different points of view. As long as they are considered as natural processes co-acting to sculpture the land surface, they are the subject of geological studies. Geologists study sliding phenomena with respect to the causes of their origin, their courses and the resulting surface forms and regard them as among the significant exogenic denudation processes. The approach of engineers and engineering geologists is different. They investigate the slopes from the point of view of the safety of the structures to be erected on them. Therefore they endeavour to identify those slopes that are susceptible to sliding where the equilibrium conditions might be disturbed by interference, and to determine the maximum permissible inclination of excavated slopes. Using reliable methods for observation and measurement of slope stability they have to propose and develop suitable measures both for the prevention of further slope movements and for the control and correction of landslide events. The quantitative investigation of slope stability was evoked by the necessity of constructing high fills and excavating deep cuttings for railways, highways and canals. The disastrous landslides on the Swedish railroads brought about the establishment of a special Geotechnical Commission in 1914, whose studies laid the foundations for a new scientific discipline, namely soil mechanics.

In landslide studies the best results can only be obtained by a combination of both these approaches. The quantitative determination of the stability of slopes by the methods of soil mechanics must be based on a knowledge (a) of the geological structure of the area, (b) the detailed composition and orientation of strata, and (c) the geomorphological history of the land surface. On the other hand, geologists may obtain a clearer picture of the origin and nature of sliding processes by checking their theories against the results of static analyses and soil and rock mechanics research.

From what has been said above, it follows that the study of sliding phenomena is of theoretical and practical importance both for the engineer and the geologist, as recognition of the causes, nature and development of landslides makes it possible to appreciate the extent of any danger and find an adequate solution to the control and correction of sliding areas. Landslides that are not anticipated or not well understood may endanger engineering projects and human lives.

1.2 Economic significance of landslides

Large-scale landslides in inhabited regions have often produced disastrous effects on the natural environment and man-made structures. The scale of damage caused by an earthquake-triggered landslide (this is invariably the most troublesome type of slope failure) is demonstrated by the catastrophe that totally destroyed the region of central Java A. D. 1006 (van Bemmelen in Holmes 1966). The slumping of a large part of the active Merapi volcano resting on weak Tertiary sediments was most likely caused by an earthquake (Fig. 1 – 1). The enormous mass of displaced material impounded a large deep lake, which flooded an extensive densely populated area. The destruction was exacerbated by a powerful eruption of the volcano induced, as is believed, by the release of pressure at depth after the partial destruction of the cone. This unpropitious interplay of earthquake, landslide, flood and volcanic eruption ruined the Hindu cultural centre of the country.

A disaster of similar extent is known from more recent times. In 1920 the Kansu province of China suffered a great earthquake, which induced the sliding of thick loess deposits over an area of 160 × 480 km. The shocks disturbed the consistency of the loess, which in its pulverized state moved into adjacent valleys burying cave dwellings and villages. About 200,000 people were killed (Close, McCormick 1922).

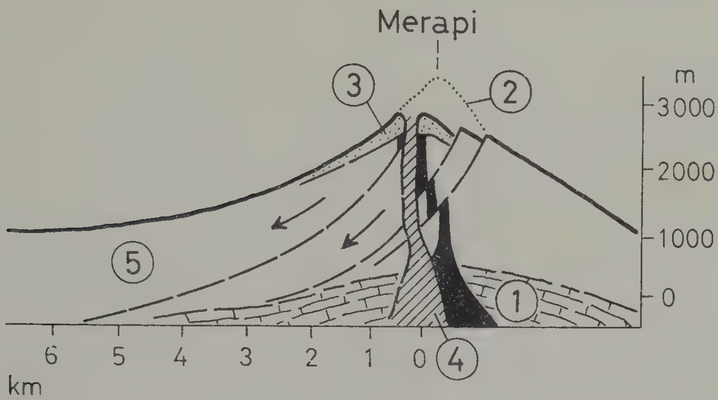

Fig. 1–1. Diagram showing the slumping of part of the cone of the active volcano Merapi in central Java; 1 — Tertiary sediments, 2 — former summit, 3 — present summit, 4 — active vent, 5 — slipped slope (after van Bemmelen in Holmes 1965).

These two and several other examples are exceptional in their extent and impact but even minor movements, particularly in areas of high landslide incidence, are of economic importance. The types of damage they can produce are listed as follows and exemplified in the text below.

Slope movements may threaten:

(a) single buildings, villages or entire towns;
(b) agricultural and forest land;
(c) the operation of quarries and exploitation of mineral deposits;
(d) communication routes and tunnels under construction or in use;
(e) hydrotechnical works such as dam construction, diversion canals, penstocks;
(f) reservoirs and lakes; material slid down the slopes increases the rate of silting up and breaks up the banks by the destructive waves that arise;
(g) water, sewage, gas conduits, telephone and electrical lines, submarine cables.

An example of an indirect adverse effect of slope movement is the situation in which a landslide blocks a valley, thus giving rise to a temporary lake, which then endangers the downstream reaches with flooding.

(a) In Europe, the Alpine countries have experienced many disastrous landslides and rockfalls, and numerous case histories have been referred to since Roman times. One of the oldest historical reports describes a large rockfall which in the year 563 destroyed the community of Taurentunum on the bank of Lake Leman. The fallen rocks blocked up the Rhône valley, producing a large wave in the lake which devastated the banks (Heim and Buss, 1881). In 1584, a large landslide on the slope of Tour d'Ai above the Rhône valley wrecked the community of Yvorne and more than 300 lives were lost (Heim 1932). Another highly damaging event was the slide of Tertiary conglomerates on the slope of Rossberg, Switzerland, in 1806, which destroyed the village of Goldau and took 457 lives (Fig. 1—2).

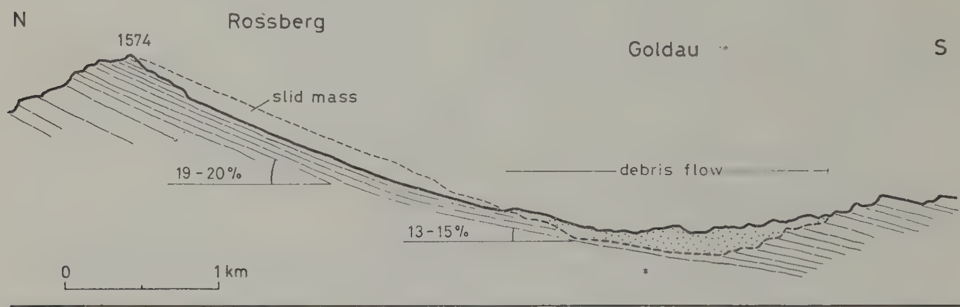

Fig. 1–2. Sliding of Tertiary conglomerates along bedding planes, which in 1806 destroyed the township of Goldau in Switzerland (from Heim 1919).

The slides that occur in the sensitive clay areas of Scandinavia and Canada often have tragic effects owing to the appalling suddenness of the motion. One of the largest landslides of this type occurred near Vaerdalen, north of Trondheim in Norway in 1893 (Holmsen 1953). A layer of sensitive clay of marine origin was laid bare by stream erosion and the liquefied clay, totalling 55 million m^3 in volume, flowed down into the Vaerdalselven river-valley within 30 minutes. The dense liquid covered an area of 8.5 km^2. The temporary lake created by the damming of the valley inundated 3.2 km^2. The disastrous landslide destroyed 22 farms and 111 persons were killed (Figs. 1—3, 1—4).

Fig. 1–3. Sketch map of the large landslip in Vaerdalen in 1893 (Holmsen 1953 ; 1 — head-scarp area, 2 — Vaerdalselven river-valley filled with liquid clay, 3 — temporary lake created by the damming of the valley, 4 — farms.

Fig. 1-4. Landslide in Leda clays at St. Jean Vianney near Quebec (Canada) in 1971. Forty houses were demolished and 31 people killed. (Courtesy Canadian Research Council).

Sliding is also a common phenomenon in Asia Minor, where the liability to sliding is enhanced by the seismicity of the region. The earliest record reporting the partial destruction of the town of Sardis by an earthquake-triggered landslide in A. D. 17 comes from Tacitus (see Chapter 2.2) (Olson 1977).

A classic example of catastrophic slope failures in North America is given by the rockslide which destroyed part of the town of Frank in Canada in 1903 (Fig. 1–5). Within two minutes about 30 million m^3 of Carboniferous limestone broke away from the mountain face burying part of the town as well as an industrial plant and a long railway track; at least 70 lives were lost (McConnel and Brock 1904). The slide was originally grouped with rockfalls since the movements was thought to occur down a set of steep joints. Further investigation and detailed mapping, however, have shown that the site of failure was the steep limb of the Turtle Mountain anticline thrust on the Mesozoic sandstones and shales. The limestones slipped along bedding planes oriented parallel to the scarp; therefore, the term "rockslide" is more appropriate (Cruden and Krahn 1973). The movement was probably triggered by a deformation caused by operations in the coal mine at the foot of the slope and by the wedging effect of freezing water in fissures. However, according to stability analyses, the slope was in a critical state before the slide event occurred owing to long-term creep.

Some examples of large rockslides in the Andes of South America are described in Chapters 2.2 and 5.

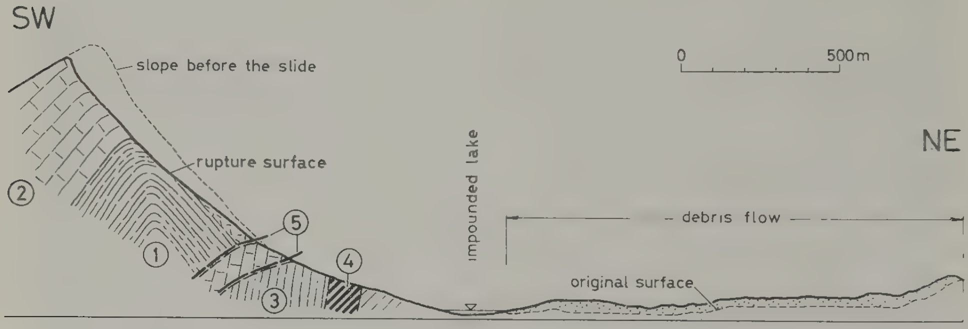

Fig. 1–5. Rockslide on Turtle Mountain in Canada, which destroyed part of the town of Frank; 1 — shaly limestone, 2 — solid limestone and dolomite (Carboniferous), 3 — sandstone and shale (Jurassic), 4 — Cretaceous sandstone and shale with coal seams, 5 — faults, disturbed zone (Cruden et al. 1978).

In Czechoslovakia landslides are not uncommon and have produced quite a lot of damage. A few of the major ones are briefly mentioned below.

In 1820 the village of Staré Stranné near Žatec (western Bohemia) was destroyed, including the church and school building. The settlement was built on the slope of the river Ohře valley made up of Neogene clays. The movements were repeated in 1872, 1882 and 1885 when the rock mass partly filled the stream bed (Pašek 1974).

On the slope of Hazmburk (north-western Bohemia), part of the village of Klapý was damaged in 1882 and 1898–1900 by sliding basalt scree and Cretaceous marls (Woldřich 1899). The movements recurred in 1939 (Stejskal 1939, Špůrek 1969). At the foot of Mužský Hill near Mnichovo Hradiště (north-eastern Bohemia),

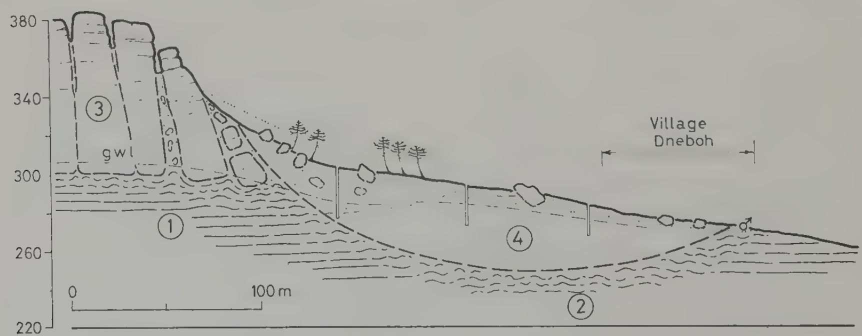

Fig. 1–6. Slide of Cretaceous marls and detritus at the foot of Mužský Hill in Bohemia in 1926 (Záruba et al. 1966); 1 — marls (Upper Turonian), 2 — moulded marls, 3 — sandstones (Senonian), 4 — slipped moulded marls.

a large slide of sandstone debris and Cretaceous marls destroyed the village of Dneboh in 1926 (Záruba 1929) (Fig. 1–6). In spring 1940 a major slide buried and wrecked the community of Dolní Týnec in northern Bohemia (Keil 1951).

Landslides occur very frequently in Moravia and Slovakia in the flysch regions of the Carpathian Mountains. In 1920 an earthflow near Hošťálková (eastern Moravia) damaged two villages; another which occurred near Dubková in the Lysá Pass, partly wrecked the community of Hlboká (Záruba 1938); a large slide near Riečnica in the Orava area partly demolished three villages in 1962.

Very serious in its effects was the landslide at Handlová in 1961, which ruined 150 houses (sect. 5.1.4), interrupted the main road, the water-supply conduit and the high-tension electric line, and threatened the railway and a newly-built part of the town. All these landslides caused great damage, but thanks to the comparative slowness of the sliding movements, no lives were taken.

(b) The depreciation of agricultural land as a consequence of sliding may also be considerable. The irregularly hummocky surface of the ground and the deep fissures interfere with cultivation and make the use of machinery impossible. In addition, slope movements may bring forth unfavourable changes in soil conditions through removal of the fertile upper layer and exposure of the barren lower layers. Where there are major disturbances, the land cannot even be used as pasture because the fissures, partly covered with vegetation, are dangerous both for people and cattle.

No less serious is damage caused by landslides in wooded areas. The yield of forest stands may be reduced with respect to the quantity or quality of the timber. Working and transport operations are likewise made difficult owing to the disturbance of the terrain. Major slope movements result in a complete extirpation of forest growth with trees uprooted or their roots drying out. Afforestation on wetted clayey soils is very difficult and usually requires costly drainage of the area and total recultivation.

The amount of damage to agricultural and timber land in Czechoslovakia is indicated by the extent of the slide-disturbed areas. The registration of landslides carried out in 1961–1962 revealed the following figures:

	Number of slides	Area in hectares
Bohemia and Moravia	4,792	30,264
Slovakia	4.372	29,136
total	9,164	59,400

Of this area, 35,000 hectares, or 59 per cent, is agricultural land and 13,500 hectares, or 23 per cent, is forest but the total area of woodland damaged is greater than this. Owing to the short period available for registration not all landslides, particularly those on the less accessible mountain slopes, could be recorded.

(c) Landslides threaten and make difficult operations in quarries and loam pits. On the other hand, ill-founded and irresponsibly operated quarries may endanger the stability of the whole slope. Thus, for instance, the quarrying of basalt in the České středohoří Mts. is frequently rendered difficult because of the sinking of marginal blocks into soft underlying rocks (Fig. 1–7). On opening the quarry in a sunken block, the floor must be gradually raised to a higher level.

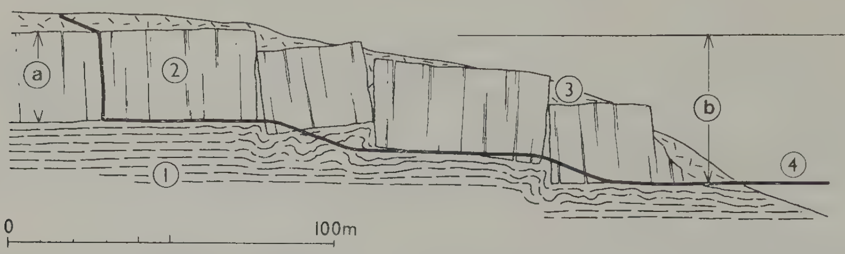

Fig. 1–7. Marginal blocks of a basalt flow (2) are disturbed by slope movements. The apparent thickness of the basalt flow (b) is greater than the true thickness (a). 1 — Cretaceous marls, 2 — basalt flow, 3 — debris, 4 — bottom of the quarry.

Considerable damage was caused by excavation of a loam pit in the Dřevnice valley (eastern Moravia). The undermining of the built-up slope disturbed its stability and the progressive backward caving of the scarp threatened a public building standing 140 m from the original edge of the loam pit. Underpinning of the building and correction of the slope were necessary. Work in the brickyard had to be stopped. Included in the total damage brought about by the reckless working procedure was a depreciation of building-plots on the slope above the brickyard (Fig. 1–8).

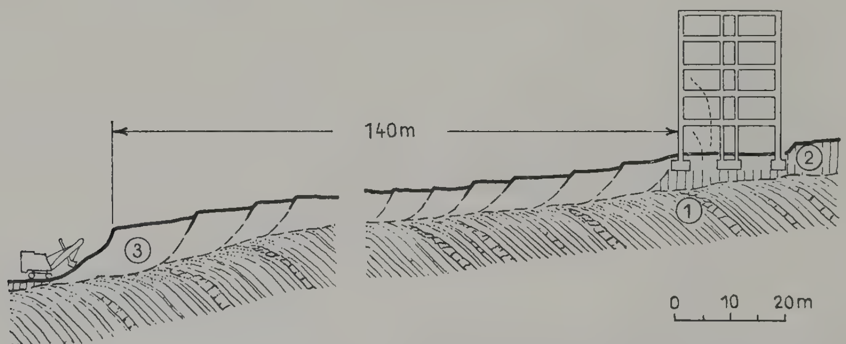

Fig. 1–8. Injudicious opening of this loam-pit at the toe of the slope provoked a landslide which threatened the stability of the building; 1 — argillaceous shales and sandstones, 2 — slope loam and debris, 3 — loam and debris disturbed by sliding.

Disastrous rocklides are known to have been caused by ill-founded quarries. Pits on talc schists opened on the slope of Monte Conto, north of Lago di Como gave rise to a rockfall that destroyed the town of Plurs and buried more than two thousand inhabitants (Heim 1932). The well-known rockfall near Elm in the Swiss Alps in 1881 was also caused by the opening of a quarry on roofing slate. In a few minutes more than 10^6 m^3 of rock material slid down; eighty-three houses were demolished and 115 lives were taken (Heim and Buss 1881).

Rockslides may also interfere indirectly with the exploitation of mineral deposits. Thus, for example, coal mining in open quarries near Dřínov at the foot of the Krušné hory Mts. (NW Bohemia) is obstructed by a large rockslide involving about $2 \times \times 10^7$ m^3 of bouldery gneiss debris, which slipped onto the Tertiary coal bearing series. The overburden that must be stripped is thus greatly enlarged. The accumulation of gneiss debris was found to be of such a thickness (up to 70 m) that at first it was considered to be rock in situ, but the drill holes have shown that the coalfield continues beneath the gneiss debris and contains a workable coal seam. A cross section of the rockslide and the buried part of the coal deposit are shown in Fig. 1–9.

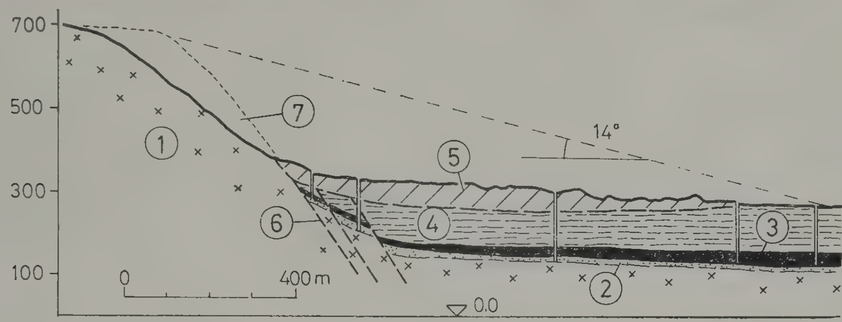

Fig. 1–9. A rockslide at the foot of the Krušné hory Mts. has covered part of the coal basin, increasing the thickness of the overlying material that must be stripped off before mining (after Špůrek 1974); 1 — gneiss, 2 — sandstone and basal detrital sediments, 3 — coal seam, 4 — overlying clay, 5 — rockslide debris, 6 — faults, 7 — slope surface before rockslide.

(d) Highways traversing areas of high landslip susceptibility are frequently interrupted by landslides, whether natural or artificial if the stability of slopes was disturbed during their construction. Several examples of damage to roadways are dealt with in Chapters 5 and 11. Landslides produce both direct and indirect damage; they endanger or even obstruct traffic and considerably increase road maintenance costs. Slope failures are often caused by the construction of access roads in the neighbourhood of deep excavations for dam abutments (Fig. 1–10).

Sliding movements create serious problems in the construction and operation of railways. The building of a railway line across a potential slide area must avoid disturbing the slope stability by deep cuttings or high embankments; also, the construction procedure often has to be adapted to the properties of the rocks. When the railway line is in operation, the trains should slow down in dangerous sections. Continuous maintenance of the slopes is necessary and the execution of remedial measures may put the line temporarily out of operation. The railway line on the seashore near Folkestone in England, for instance, has been interrupted several times by landslips (Ward 1945).

Fig. 1–10. Highway interrupted by landslide caused by excavation for dam abutment (photograph by Záruba).

Occasionally, where there has been ever present danger of sliding movements demanding high maintenance costs a railway line has had to be relinquished. In Bohemia, the Žabokliky—Březno line (Žatec area) was discontinued after six years of operation. The traffic on this line, which ran on the valley side of the Ohře river, was too small to cover the cost of the reconstruction and maintenance work. Benson (1940) reported a similar case in New Zealand.

Considerable damage to railway lines may be caused by slope movements in opencast mines (Fig. 1—11).

Landslides also present great problems in tunnel construction. When the greater part of the Unterstein tunnel near Salzburg (Austria) had been lined and the last sections were being excavated, chloritic schists on the slope slipped down, causing the tunnel to cave in. The original line had to be abandoned and a new tunnel was driven deeper into the mountain (Wagner 1884).

A noteworthy tunnel failure occurred in New Zealand; a 175 m long railway tunnel built in 1878, had to be abandoned and the line rerouted in 1935 (Benson 1940). The tunnel was constructed in Miocene sandstone overlying claystones dipping seawards at 15°. It was discovered that the tunnel crossed the upper part of a large

Fig. 1–11. A railway line interrupted by a landslide; Marica-Iztok brown-coal basin in Bulgaria (photograph by Rybář).

rockslide and was traversed by several cracks separating the individual displaced blocks (Fig. 1 – 12). Geodetic survey has shown that the blocks continued to settle and rotate as a result of which the height position of the rails had to be continuously adjusted. In addition to these movements, sliding of the whole area has been revealed, the rate of this depending on the rainfall amounted to 2 – 7 cm per month in the years preceding the dereliction of the tunnel.

Figure 1 – 13 shows the collapse of a 187 m long railway tunnel in Japan. The tunnel was driven in Neogene mudstone and sandstone in the side of the Shinano

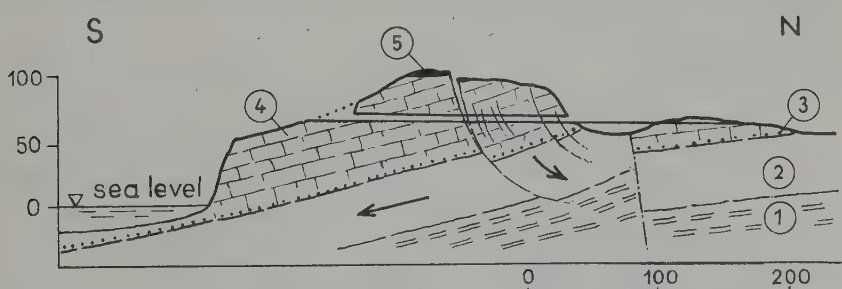

Fig. 1–12. Longitudinal section of the railway tunnel in New Zealand which was damaged by a landslide in 1935 (after Benson 1940); 1 — glauconitic claystones (Upper Cretaceous), 2 — Tertiary claystones, 3 — glauconitic sands, 4 — crumbly sandstones, 5 — basalt.

river valley in about 1925 and in 1970 it was destroyed by a landslide. The deformations due to progressive creep were noticed for a long time and continued monitoring of the movement as well as control measures were carried out. It is noteworthy that

Fig. 1-13. The slope failure which destroyed the Takabayama tunnel in Japan, 1970; 1 — landslide in Neogene mudstone, 2 — railway line, 3 — highway, 4 — Shinano river (Japan Society of Landslides 1972).

the time of slope failure was forecast precisely on the basis of these observations and movement analysis.

(e) Slope movements of different kinds have caused heavy losses to dam constructions, these movements having occurred mainly during excavation works. Thus, for instance, the right abutment of the Grand Coulee Dam on the Columbia River in the U.S.A. had to be stabilized during construction by freezing the young sediments that were moving downslope. The foundation excavations for the Fork Site Dam on the San Gabriel River in California brought about such a large rockslide along a fault zone in a granite mass that the dam site had to be abandoned (Záruba 1934) (Fig. 1 – 14).

Fig. 1–14. Rockslide in apparently firm granite, which caused the Fork Site locality in California to be abandoned (photograph by Záruba).

In Slovakia, the stability of the valley slope was disturbed during excavation of the foundations for the Dobšiná Dam. Fractured gabbro-diorite began to slide on Carboniferous graphitic shales dipping 35° downslope. Fortunately, the sliding was arrested by supporting the loosened rock upon completed dam segments by means of heavy frames, thus allowing completion of the dam construction (see Chapter 10).

The question of slope stability plays an important role in the construction of navigation and diversion canals. The building of canals often requires deep excavations which may give rise to extensive and very dangerous slides. Major landslides occurred, for instance, on the Panama Canal, in the Culebra cutting (Voight 1978).

In Slovakia, slope sliding was a serious problem in the construction of the diversion canals on the river Váh; in some sections this factor determined the design of the whole project. Thus, for example, the axis of the diversion canal between Krpeľany, Sučany, Lipovec (Záruba, Mencl 1958) had to be modified because of deep-reaching landslides in Neogene sediments at the northern margin of the Turiec depression. The diversion canal near Mikšová on the Váh was also threatened by a fossil landslide which was reactivated by the excavation work (sect. 5.2.2.).

Landslides and slope movements may also adversely affect penstocks and pressure conduits feeding water to power plants. Benson (1946) described such a case in New Zealand. The pressure conduit founded on a slope consisting of deep-weathered chloritic schists, displayed deformations in the anchoring blocks soon after the structure was completed. As the deformation reached 10 cm after only three months, the conduit had to be transferred from the slope surface into a gallery, which had to be driven deep in the slope, since geophysical investigation had shown that fresh rock occurred at a depth of 60 m. The longitudinal profile of the conduit had to be deflected at several points (Fig. 1 – 15).

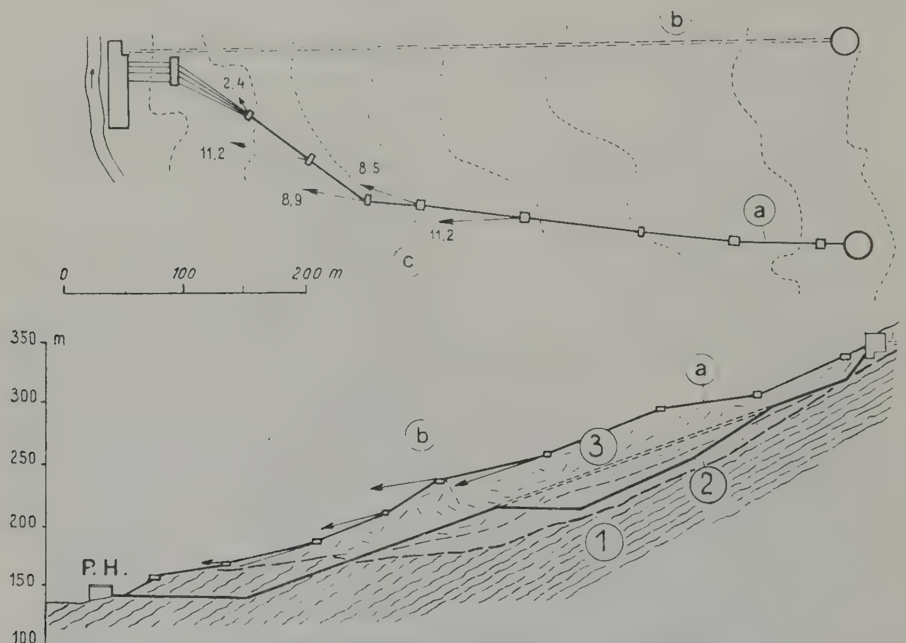

Fig. 1–15. Deformation of penstock due to siting on an unstable slope; the pipe-line had to be relocated underground (Benson 1964); 1 — chlorite gneiss, 2 — weathered gneiss, 3 — slope debris, a — original position of the penstock, b — penstock relocated into a gallery, c — movements of blocks in cm.

An ancient landslide caused severe problems during the construction of the pressure tunnel for the Latschau power plant at Lünersee, in Austria. Water from the Lünersee reservoir should have beeen conducted to the power plant by an inclined pressure tunnel. However, geological survey of the area showed that the slope above the power plant was disturbed by a Pleistocene landslide, and therefore a section of the conduit had to be located on the slope surface so as to shorten the section built in the slid rock mass. The shape of the displaced block was assessed by a series of borings and exploration drifts. In the opinion of Austrian geologists (Mignon 1962), a voluminous complex of crystalline schists slipped along an ancient tectonic surface onto an old moraine at the foot of the slope following the retreat of a glacier. The driving of the pressure tunnel was extremely difficult in this section, especially at the base of the slipped blocks where crushed Triassic sandstones and morainic deposits were encountered.

(f) Sudden rockslides on seashores may have indirect, yet disastrous effects. In Norwegian fiords, rockslides often give rise to high swells of up to several tens of metres which threaten the inhabited coast. In 1936 a rockfall, 10^6 m^3 in volume occurred near Loen and produced a 74 m swell in the Nordfjord; in this catastrophe 73 lives were lost (Bjerrum and Jörstad 1966).

One of the worst slide-caused disasters known occurred in the Vaiont reservoir in the Italian Alps in 1963. A complex of about 260 million m^3 cf Jurassic and Cretaceous limestones suddenly slid down the slope of Mount Toc into the reservoir impounded by the 265 m high dam. The mass abruptly filled the greater part of the reservoir, causing a surge of water higher than 100 m which overflowed the crest of the dam, destroyed the township of Longarone, and damaged other communities in the Adige valley. Almost 2,000 people lost their lives (Selli et al. 1964).

Fig. 1–16. Banks of the Nechranice reservoir in NW Bohemia affected by sliding (photograph by Záruba).

A series of dangerous slides were caused by lowering the level of some Alpine lakes after their incorporation into the new hydro-electric schemes. During the construction of the Davos power plant in Switzerland, for example, the lowering of the Davos Lake level by 11 m (Moor 1923) triggered a slide of 900,000 m^3 of material of an alluvial cone. The abrupt movement of such an enormous mass produced a violent surge which broke an ice cover 80 cm thick on the lake into small floes, and caused heavy losses on the shores. The artificial lowering of the level of Lake Spullersee in the Tyrol likewise resulted in the downslipping of banks formed of pelitic sediments and deltaic deposits at the mouth of brooks (Ampherer, Ascher 1925).

The life of some reservoirs is shortened because of banks collapsing and therefore adding to the rapid silting of the reservoir. This is especially true in valleys where the slopes are disturbed by ancient landslides or are susceptible to sliding, as is the case in some Alpine and Carpathian regions.

In Bohemia, extensive sliding disturbs the banks of the Nechranice reservoir, which are formed of Neogene clays and silts. The large-scale slope movements are repeated after every drawdown (Fig. 1–16).

Major landslides are indirectly responsible for catastrophic events that occur when the slipped material blocks a river and holds back the water. Such barriers are generally not strong and collapse as soon as the water reaches the top causing disastrous floods. One of the largest catastrophes of this kind occurred in the southern Alps in the sixteenth century. In 1512 a rock slide of 150 million m^3 dammed the Brenno valley near the township of Biasco creating a lake of more than 50 m depth and with about 200 million m^3 capacity. Two years later the dam collapsed and the flood caused heavy losses downstream and on the banks of Lago di Maggiore. According to historical data, more than 400 houses were destroyed and the loss in lives exceeded 600 (Heim 1932).

A large temporary lake was caused by the blocking of the Rhine valley by an interglacial rockslide near Flims. The lake level rose to more than 600 m above the valley floor, but now the lake no longer exists, as the powerful stream eroded the valley almost to its original level (see Chapter 5).

Extensive landslides of this kind were also recorded in the Himalayas. Thus in 1893 a temporary 320 m high dam was created on the upper Ganges. The following year the dam was overflowed and a disastrous flood occurred. Although whole villages and towns were swept away, no life was lost since timely precautions were taken, including the installation of a telegraph line between the dam and the lower parts of the valley (Legget 1962).

One of the largest slope failures of this type occurred on April 25, 1974 in the Mantaro river valley in Peru (Fig. 1–17). A huge complex of Permian sandstone with marl interbeds slipped along bedding planes from the upper part of a steep slope. A debris flow over 8,000 m long blocked the canyon forming a temporary lake of about 670 million m^3 capacity. The volume of the rock mass was estimated at 1,000 million m^3. With a vertical fall of about 1,500 m the rock movement reached

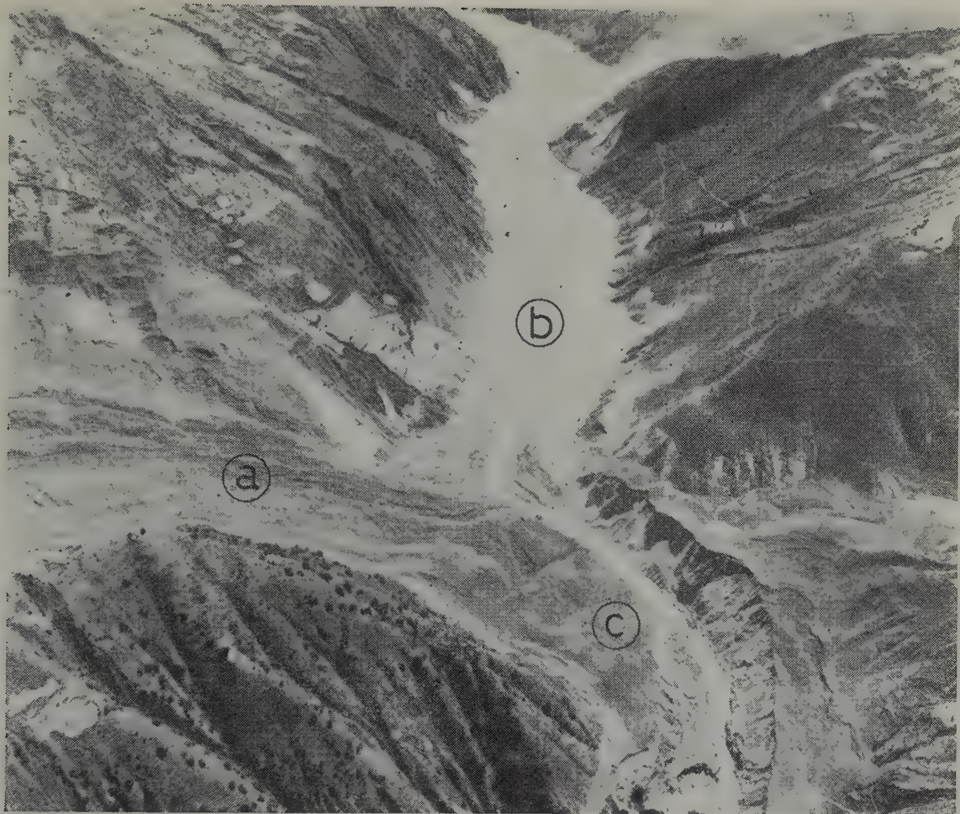

Fig. 1-17. Rockslide and debris flow on the Mantaro river in the Andes, Peru, 1974. View of breached landslide dam which ponded a temporary lake; a — debris flow, b — temporary lake, c — landslide dam with a deep scour zone (Kojan and Hutchinson 1974).

a velocity of 120–140 km/hour, and lasted 3–4 minutes. This disastrous slide destroyed several villages and 450 people lost their lives. 44 days later the debris dam was overflowed and washed away in two days. The flood wave was up to 35 m high but thanks to a timely evacuation of the valley no deaths occurred.

Six months before the slide cracks were observed in the upper part of the slope, but no earth tremors or excessive rainfall immediately preceded the movement. Since, however, the valley lies in a seismic area, it seems likely that previous vibrations brought about some loosening of the sandstone beds. A contributing factor was the high rate of river erosion which resulted in a very deep valley with steep slopes (Kojan and Hutschinson 1978).

When dams created by slipped rock masses are sufficiently large and firm, permanent lakes may be formed. Thus the lake of Klönsee in the Glarn Alps was produced by a large interglacial landslide the volume of which is estimated at 770 million m^3.

Davos Lake and a number of other lakes were also formed as a result of landslides.

Rockslide-dammed lakes have also been identified in the Carpathian region. In 1828 the slide of a rock wall formed of Dogger limestones, east of Georgheni in Rumania, gave rise to the still existing lake of Lacul Rosu. As the flooded valley was densely forested, numerous remains of trees now protrude above the water. The lake in the Blatná valley south of Lubochňa, Slovakia, was also generated by a large rockslide of dolomitic limestones (Záruba and Ložek 1966).

Many dammed lakes have gradually become silted, and alluvial deposits have formed steps in the longitudinal profiles of watercourses. In some places these steps have been preserved; elsewhere, only the remnants of lacustrine deposits have been left on slopes, the main aggradations having been removed by erosion (see Chapter 10).

The above few examples give an indication of the economic importance of slope movements and the need for their thorough investigation. In the United States attempts were made to estimate the financial losses caused by sliding events. Directly and indirectly caused losses have been estimated at about one thousand million dollars per year. Of this sum 500 million is attributable to the damage to buildings and their sites and 100 million $ to the disruption of highways and roads. The overview of world landslide disasters has revealed that they cause greater losses of life and property than any other geological catastrophe (Schuster 1978).

Chapter 2

FACTORS CAUSING MASS MOVEMENTS

It is of primary importance to recognize the conditions that cause slopes to become unstable and the factors that trigger the movement. Susceptibility to sliding is determined by the geological structure of the slope, the lithology of the rocks, hydrogeological conditions and the stage of morphological development of the area. Only an accurate diagnosis makes it possible to appreciate the extent of the danger and to propose effective remedial measures. The great variety of slope movements reflects the diversity of factors that may disturb slope stability. The most important of these are as follows:

(1) *Changes in the slope gradient:* These may be caused by natural or artificial influences (e.g. by the undermining of the foot of a slope by stream erosion, or by excavations). Exceptionally, the angle of the slope is steepened as a result of tectonic processes, such as subsidence or uplift. An increase in slope gradient produces a change in the internal stress of the rock mass and equilibrium conditions are disturbed by the increase of shear stress.

(2) *Changes in the slope height* as a result of vertical erosion or excavation work. The deepening of a valley relieves lateral stress and this in turn leads to the loosening of rocks in the slope and the formation of fissures parallel to the slope surface. The penetration of rain water is thus facilitated.

(3) *Overloading by embankments, fills and spoil heaps.* This produces an increase in shear stress and an increase in the pore-water pressure in clayey soils, which results in decreased shear strength. The more rapid the loading, the more dangerous it is.

(4) *Shocks and vibrations.* Tremors produced by earthquakes, large-scale explosions and machine vibrations affect the equilibrium of slopes on account of the temporary changes of stress that are caused by oscillations of different frequencies. In loess and loose sand shocks may disturb intergranular bonds and thus decrease cohesion. In water-saturated fine sand and sensitive clays, the displacement or rotation of grains can result in a sudden liquefaction of the soil.

(5) *Changes in water content:*

(a) Rain and melt water penetrate joints and produce hydrostatic pressure. In soils the pore-water pressure increases and consequently the shear resistance decreases. Measurements of rainfall have confirmed that recurrent slope movements occur in periods of exceptionally high rainfall.

(b) It has been found that between two beds which contact along a sliding plane there is a difference of electric potential. The increase in the water content leading to slope movement is explained as an electro-osmotic effect.

(c) In clayey rocks, the deleterious effect of atmospheric water is greater when the rain comes after a long dry period: clayey soils are desiccated and shrunken so that water readily percolates deep into the fissures.

(6) *Effects of ground-water:*

(a) Flowing ground-water exerts a pressure on soil particles which impairs the stability of slopes. Abrupt changes of water level as might occur in reservoirs, cause pore-water pressure in slopes to increase and this in turn may lead to liquefaction of sandy soils.

(b) Ground-water can wash out soluble cementing substances and thus weaken the intergranular bonds and reduce the mechanical strength of the ground.

(c) In fine sand and silt flowing ground-water flushes out fine particles and the strength of the slope is weakened by the cavities that are formed.

(d) Confined ground-water exerts an upward pressure on overlying beds.

(7) *Frost effects.* Water freezing in rock fissures increases in volume and thus tends to widen them; rock penetrated by fissures consequently shows reduced cohesion. In clays and clayey-sandy soils ice laminae are formed, which on melting enlarge the water content in the thawing surface layer. The freezing of water on the surface impedes drainage from the slope, so that the ground-water table rises and equilibrium is eventually disturbed.

(8) *Weathering*, both mechanical and chemical, gradually disturbs the cohesion of rocks. In many landslide events, chemical alterations such as hydration and ion exchange in clays are thought to have contributed to the triggering of landslides. The tendency of slopes to slide has, for example, been established in areas made up of glauconitic sands and clays.

(9) *Effects of vegetation.* The roots of trees maintain the stability of slopes by their mechanical effects and contribute to the drying of slopes by absorbing a part of the ground water. Deforestation of slopes adversely affects the water regime in the subsurface layers.

2.1 The relationship between slope movements and precipitation

Rainfall is generally accepted as one of the chief factors controlling the frequency of landslides. The magnitude of its influence depends on climatic conditions (such as the distribution of precipitation and changes in temperatures), on the topography of the area, the geological structure of slopes, and the permeability and other properties of rocks and soils.

The relationship between the amount of rainfall and frequency of landslide events has been studied by many authors. As early as 1906 – 1910 Almagia published a two-volume book on Apennine landslides, in which he listed 300 landslides occurring between the 11th and 19th centuries and drew correlations between their occurrence and the regional precipitation levels. Lichkov (1938) conducted a statistical analysis of about 350 slope failures and their relationship with rainfall and snow melting in the Kiev area. Schmassmann (1953) studied the landslides which occurred in the Basle Canton and assessed their dependence on the pattern of precipitation, using for this purpose cumulative curves for the successive twelve-month periods of rainfall. This problem has also been examined by Emelyanova (1953, 1972). The influence of precipitation on sliding activity in the Carpathian region has been dealt with particularly by Sawicki (1917), Śliwa (1955) and Jakubowski (1968).

A systematic study of this aspect of landslides by Czechoslovak geologists has been prompted by the finding that in the Bohemian Cretaceous Basin and the České středohoří Mts. increases in the numbers of landslides coincide strikingly with periods of increased rainfall. In these areas high levels of precipitation constitute a "provocative" sliding factor, according to Almagia's terminology.

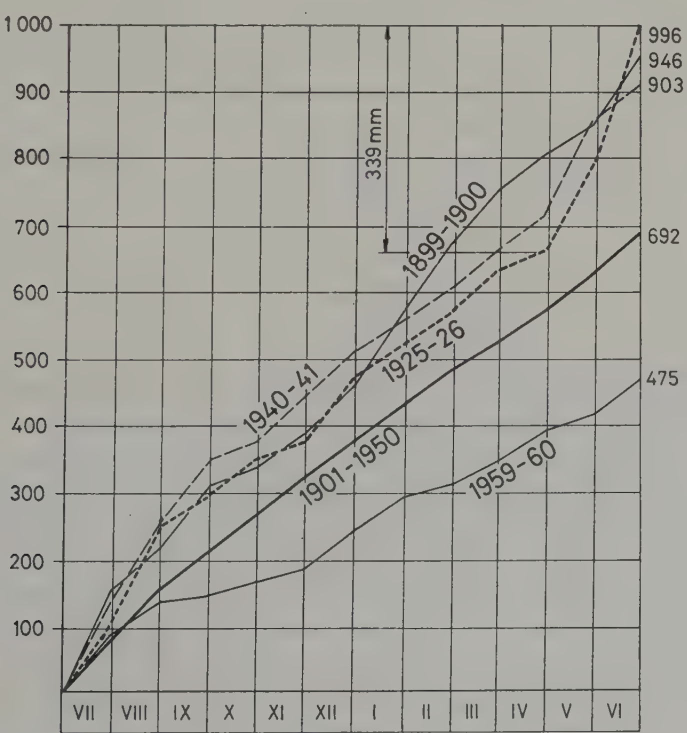

Fig. 2–1. Cumulative monthly rainfall diagram for different years, based on observations of the State Hydrometeorological Institute; data from the Turnov area in Bohemia.

The relationship between landslides and precipitation is illustrated in Fig. 2–1, which shows the cumulative curves for monthly rainfall in several years in the Turnov area (data taken from the report of the State Hydrometeorological Institute). New slope movements and the revival of old ones occurred in the years 1899 – 1900, 1915, 1926, 1941 and 1958, which were also the years of maximum rainfall. The rain maxima were 996 mm between July 1, 1925 and June 30, 1926, 945 mm in the same period of 1899–1900, and 903 mm in 1940–1941. The fifty years' mean precipitation was 692 mm. The annual period in the diagram runs from July 1 to June 30 of the following year because the precipitation of the autumn months is a decisive factor affecting ground-water conditions in spring.

Slope movements generally occur in two periods of the year: in spring after thawing and in summer (June and July) after heavy rains. The diagram shows that slope movements may be expected when a rainy spring follows a wet autumn and winter and the total precipitation of the previous 10 months exceeds 700 mm. This critical value was established for the landslide area near Přerov nad Labem (Záruba 1926) and was corroborated by subsequent observations in Turnov and its neighbourhood (Záruba et al. 1966).

In 1926, ten months of above-normal precipitation were followed by an extremely wet period in May and June (339 mm), which caused many new slope movements and a reactivation of ancient landslides within the whole of the Bohemian Cretaceous Basin. During these 12 months precipitation exceeded the 50-year average by 44 per cent and in May and June by 165 per cent.

The relationship between rainfall and the frequency of landslides is even more clearly demonstrated in Fig. 2–2 which shows successive three-year averages of

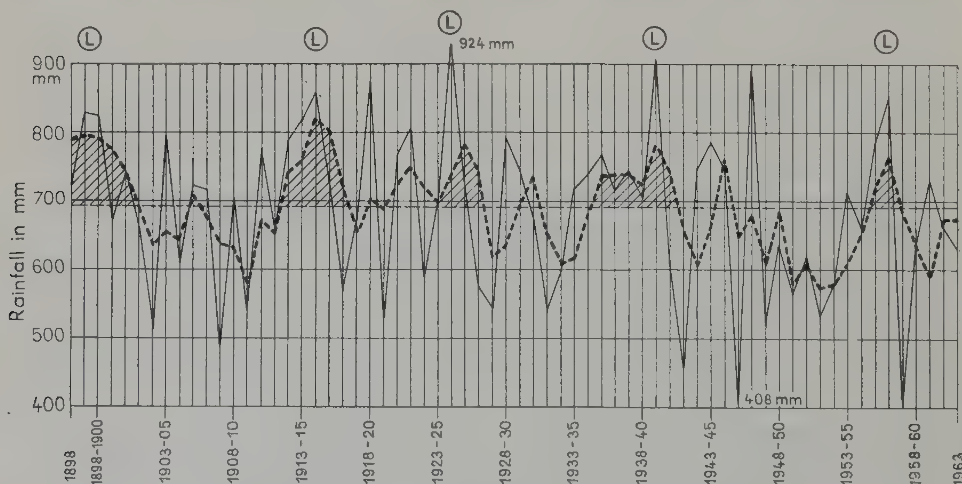

Fig. 2–2. Annual rainfall in the Turnov area between 1898–1963; dashed line — computed three-year averages. Hatched areas show precipitation higher than the 50-year average, and letter L denotes the years of frequent occurrence of landslides.

precipitation from 1897 to 1963, according to the records of the ombrometric stations in Turnov and Karlovice. In the Bohemian Cretaceous region, where the ground-water table is relatively deep, the successive three-year averages are most important for studying the effect of climate on ground-water conditions, because the latter are influenced by the precipitation of the two preceding years. The periods of above-normal rainfall (e. g. 1887–1900, 1914–1916, 1925–1927 and 1939–1941) are hatched in the diagram. In 1898–1900 the village of Klapé on the slope of Hazmburk was damaged by a succession of landslides, in 1941 the village of Horní Týnec near Litoměřice was partially buried by a large landslide, and in 1926 slope movements were reactivated in many sliding areas near Mladá Boleslav, Turnov (the village of Dneboh was destroyed), Jičín, Kladno and Přerov nad Labem (Záruba 1926).

The numbers of recorded landslides in the Bohemian Massif during several periods are given in Table 1 (after Špůrek 1970).

Table 1

Phase of landslide activity	Period	Number of recorded landslides
I	1880—1883	8
II	1897—1900	91
III	1914—1916	7
IV	1925—1927	42
V	1939—1941	49
VI	1965—1968	13

Špůrek (1970) studied the relationship between yearly and monthly climatic anomalies and the frequency of sliding phenomena in the period 1880–1968. He constructed a diagram of landslide occurrences in the Bohemian Massif in relation to the seasonal and yearly cummulative precipitation: he also plotted in the diagram the cumulative lines of annual deviation from the fifty year average. The majority of the sliding phenomena are concentrated to the rising sectors and peaks of the cumulative line of deviations. Špůrek differentiated six main climatically controlled phases corresponding with the periods of high landslide activity as shown in Fig. 2–3.

A detailed analysis of sliding events reveals differences in the sensitivity of slopes to the effect of rainfall. The interval between the commencement of above-normal rainfall and the start of slope movement varies according to the permeability of the surface rocks and the type and form (especially the depth) of the landslide induced.

The short intense rain storms that give rise to debris flows rarely occur in Central Europe. They are limited to high mountain regions in the Alps and Carpathians, where above the treeline there is an abundance of free debris with only a sparse

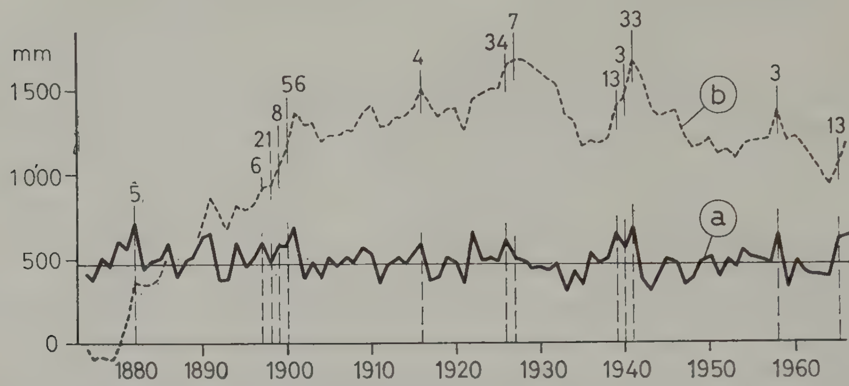

Fig. 2–3. Landslide occurrence in the Bohemian Massif shown in relation to the summed annual precipitation and the cumulative departures of the summed annual precipitation from the fifty-year average (determined for the meteorological station at Louny); a — fluctuation of summed annual precipitation about the fifty-year average (1901—1950), b — cumulative deviation of summed precipitation from the fifty-year average (1901—1950). Number of recorded landslides given above line b (after Špůrek 1970).

vegetation cover. During torrential rains the debris is washed down erosion gullies forming the "Muren".

Thus, for example, in the High Tatra Mts. numerous debris flows were triggered by heavy rain (26 mm per hour) on July 15, 1933, and this disturbed overgrown debris cones in many places. On that day 45.8 mm of rainfall was measured between 6 p.m. and 8 p.m. at the Starý Smokovec ombrometric station. This unusually intense rain shows clearly in the hourly rainfall diagram in Fig. 2—4. In the Figure several plots of maximum diurnal rainfall in the High Tatra Mts. are shown (adapted from the data of the Hydrographical Department in Bratislava). The maximum 24-hour rainfall amounting to 62 mm was measured on July 16—17, 1934, but owing to its low intensity (5—8 mm per hour) it did not induce any debris flow.

The influence of climatic conditions on sliding phenomena in Central Europe, where precipitation is distributed throughout the year, cannot be compared with climatic effects where rainfall is seasonal and restricted to shorter time intervals, as for example in California and Japan. In California, most of the existing landslides in the areas formed of Tertiary sedimentary rocks will be set in motion whenever the seasonal rainfall amounts to about 250 mm (Radbruch-Hall and Varnes 1976). If the rains continue to increase slowly, larger and deeper areas will gradually become saturated and more slope failures such as slumps, large earth-flows and slump-flows will develop or become reactivated.

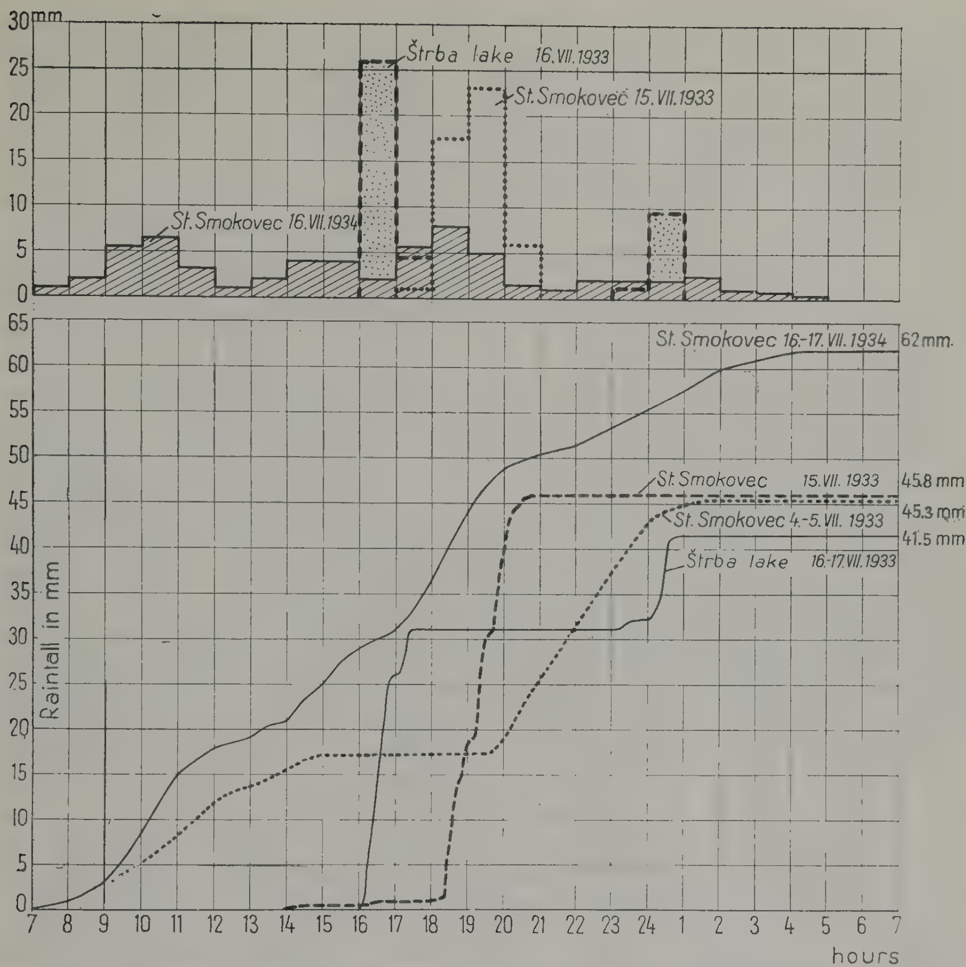

Fig. 2-4. The intensity of rainfall is a decisive factor in the development of debris flows. A heavy downpour on July 15, 1933 triggered numerous debris flows in the High Tatra Mts.

The relationship between renewed sliding movements and excessive rainfall is well demonstrated in the case of a deep landslide in Portland (Oregon), which affected the area of the local zoo. The slide developed in decomposed basalt (stiff clayey silt). In the upper part of the slope a curved slide surface developed, which in the lower part followed the nearly flat surface of the dense weathered basalt. The landslide was about 200 m long, 300 m wide and the layer of clayey material involved in the slide was approximately 25 m thick. It is an old landslide area, the stability of which was disturbed by excavation for a new road in 1957. In 1958–1959 high rainfall caused a new movement in which a distinct marginal crack delimiting the head scarp was formed. Between 1963 and 1970 the landslide was studied in detail, particularly with respect to the relationship between recurrent movements and rainfall.

From Fig. 2–5 it is apparent that the movement was reactivated when the winter rains exceeded 200 mm per month. The landslide was stabilized by a system of relief wells and drainage borings as well as a rip-rap buttress at the front of the landslide (Radley-Squier and Versteeg 1971).

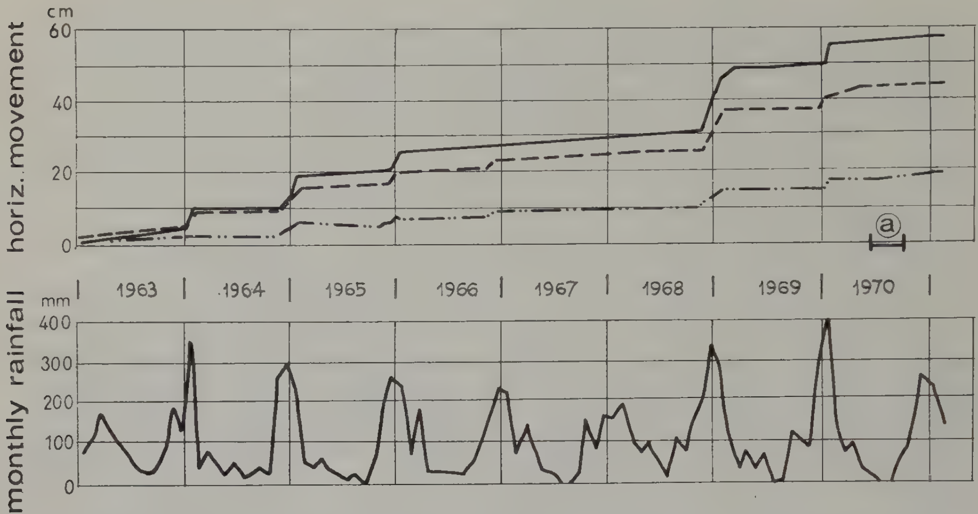

Fig. 2-5. Relationship between horizontal movement of Zoo slide in Portland, U.S.A., and rainfall intensity; a — period of buttress construction (Radley-Squier and Versteeg 1971).

Similar investigations in Japan have shown that the number of slope failures increases abruptly when the cumulative precipitation over the 1–3 days of heavy rain exceeds 150–200 mm and has an intensity of more than 20–30 mm per hour (Onodera et al. 1974). Since a daily rainfall of 500 mm reaching intensities of 100 mm per hour is not rare in Japan (according to 50-year ombrometric records), it is no wonder that landsliding there is a very serious economic problem. For example, the disastrous landslide that disrupted the railway line near Shigeto (Kochi) in 1972 was triggered by rainfall amounting to 742 mm in one day (Fig. 2–6); the average precipitation in this area was 3389 mm per year in the period from 1931 to 1961 (Japan Landslide Society 1972).

Lumb (1975) reported similar values for the Hong-Kong area, where slope failures (more than 50 slips per day) occur when daily rainfall exceeding 100 mm follows two weeks of precipitation totalling more than 350 mm.

A systematic analysis of ombrometric records may help to predict the reactivation of slope movements in landslide-susceptible areas and thus reduce the threat to human life and property.

Fig. 2–6. The Shigeto railway station in Japan was partly buried by a landslide in 1972. The locomotive and two carriages were thrust into the river Ananai. The slope failure was triggered by torrential rain amounting to 742 mm per day (courtesy of Asahi Shinbun).

2.2 Landslides in seismic regions

Central European geologists have somewhat underrated the role of earthquakes in landslide phenomena, probably because there are practically no major earthquakes in these countries. But if the seismically active regions of the world are taken into consideration, it is clear that earthquakes constitute one of the most important landslide-inducing agents.

During earthquakes with intensities[1] higher than VIII° according to the Inter-

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[1] The intensity of an earthquake is measured on the basis of the effects it produces at the earth's surface. The scale commonly used for its determination is the Mercalli-Cancani-Sieberg scale, which has twelve grades: I (almost imperceptible tremors) to XII (earthquakes causing extreme damage).

The magnitude is a measure of the strength of an earthquake or strain energy released by it. The magnitude is determined from the maximum amplitude of the seismic waves; the scale of its values ranges from zero to 8—8. 6.

national MCS scale (magnitude over 6.5), particularly those occurring in mountain regions, the largest damage to property and the greatest loss of life are caused by seismo-gravitational phenomena (rockfalls and landslides). The movements of the earth's crust are accompanied by major deformations of the ground surface and destruction of mountain slopes. Shocks and vibrations influence the stress state in rocks: shear stress usually increases and the shear strength is occassionally decreased, which results in the disturbance of the stability of slopes.

In Europe, one of the seismically most active regions is Calabria in Italy. The earthquake of February 5–6, 1783 had disastrous effects. Numerous mass movements involving slumps, slides, falls and the spontaneous liquefaction of loose material destroyed villages and forests and diverted or blocked streams. The displaced masses gave rise to a large number of lakes, 215 of which are plotted on a map contemporaneous with the event. It is interesting to note the impact this disaster had on the living environment; the marshy area became a source of malaria and required extensive drainage and reclamation work (Cotecchia and Melidoro 1974).

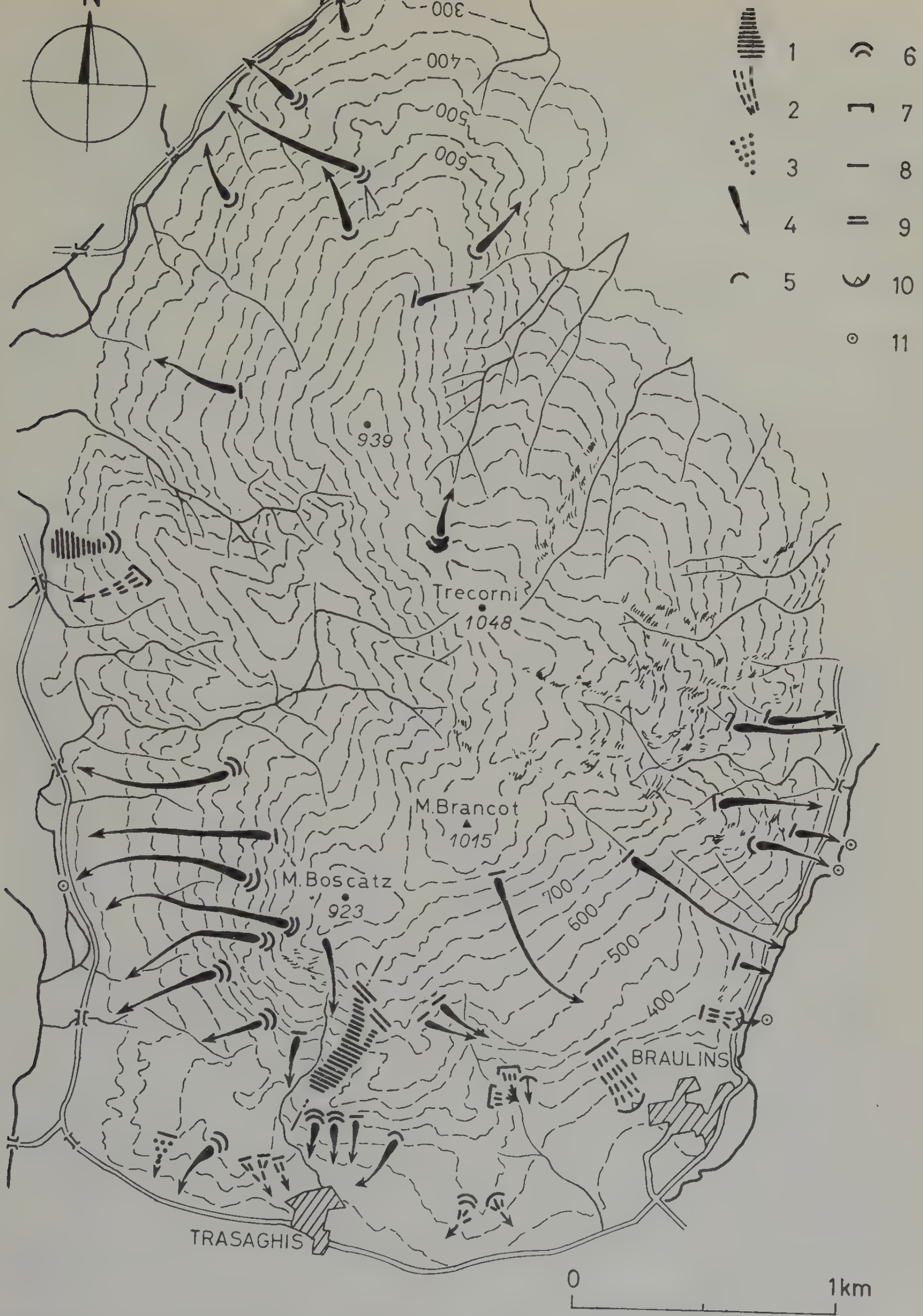
Another example may be cited from the Pre-Alps of Arzino, where a severe earthquake in March 1928 set in motion more than 400 landslides and rockfalls.

A catastrophic earthquake causing the deaths of nearly 1000 people and the destruction of many villages affected the nearby pre-Alpine area of Friuli in 1976. The main shock of magnitude 6.4 with its epicentre west of Gemona in the Tagliamento river valley occurred at the beginning of May and another occurred on September 15. The epicentre of the latter was about 7 km to the north of the first and its magnitude was 6.1. Almost the whole of the epicentral zone was mapped by M. Govi (1977), who established more than 250 rockfalls and landslides.

The Friuli area is formed of sedimentary rocks ranging from Late Triassic to Quaternary in age. Lithologically, these are dolomites, limestones and calcareous-marly flysch sequences. Early Quaternary massive layers of breccia and coarse-grained conglomerates are found in the Tagliamento valley itself. The area underwent strong Alpine deformation which is shown by the intense folding and faulting.

Most of the mass movements triggered by the Friuli earthquake were rockfalls which occurred at the sites of old slope movements (Figs. 2–7, 2–8); newly formed landslides were relatively few. This led the author to distinguish "areas of high geoseismic hazard", which are in temporary equilibrium but are repeatedly subject to slope disturbances caused by earthquake shocks. The main factors influencing the rockfalls were the weakening of the rock due to tectonic disturbance and the steepness of the slopes as determined by structural and lithological conditions.

Fig. 2–7. Part of the map showing slope movements triggered by an earthquake in 1976 in the Friuli area of Italy (compiled by Govi 1977); 1 — rockfalls due to reactivation of recent landslides, 2 — reactivation of old events, 3 — movements in previously undisturbed zones, 4 — falling of



single blocks, 5 — pre-existing niche or scar, 6 — ditto, greatly extended, 7 — newly formed niche, 8 — steep surface of rupture, 9 — ditto, preexisting, greatly extended, 10 — newly formed accumulation, 11 — the furthest point reached by single blocks.

In some of the most seismically active regions of Asia Minor, many hundreds of earthquakes (many of them having catastrophic results) have been recorded since antique times. They brought about severe seismotectonic deformations of the earth's surface and caused extensive and disastrous mass movements. The oldest record

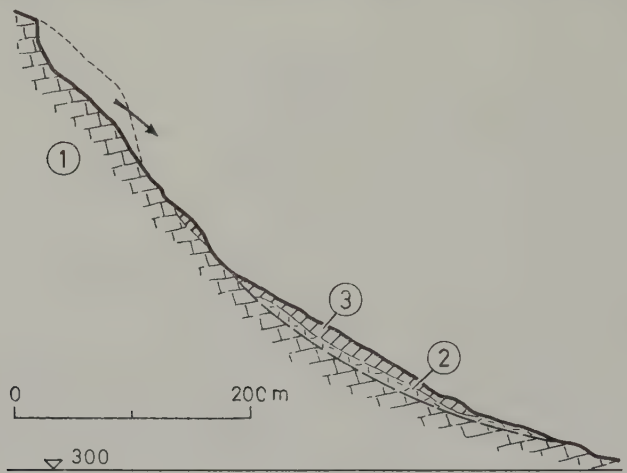


Fig. 2–8. Earthquake-triggered rockfall south of Mt. Boscatz, 1976 (Govi 1977); 1 — dolomites and dolomitic limestones (Triassic), 2 — pre-existing accumulation of rock debris, 3 — accumulation due to seismic shock.

refers to the ancient town of Sardis, which according to Tacitus had been partly destroyed by earthquake and landslides in the year 17 A. D. Since the 11th century the surroundings of Sardis have been affected by about 350 earthquakes, many of which triggered extensive sliding on mountain slopes (Olson 1977).

A violent earthquake in 1967 affected a region of West Anatolia covering about 450,000 km^2, and the landslides which ensued caused much damage, including disruption of the Istanbul – Ankara railway line in the Sakarya river valley. The shock had a magnitude of 7.1 and effected a lateral displacement of 190 cm on the Anatolian fault zone over a distance of 80 km (Ambrasseys et al. 1968).

Seismogenic disturbance of mountain slopes is intensively studied in the Soviet Union. On the basis of his study of the severe earthquake in the Gobi Altai on December 4, 1957 (magnitude 8.6) Solonenko (1972) proposed the following classification of seismogenic deformational phenomena:

A – seismotectonic type,
B – gravitational-seismotectonic type,
C – seismogravitational type.

The seismotectonic phenomena are a manifestation of crustal movements along faults, folds and flexures during earthquakes of magnitude higher than 6.5. These do not depend on gravitational forces acting at the earth's surface and the characteristic of the deformation is governed by the nature of the stress field.

The gravitational-seismotectonic phenomena (seismotectonic wedges, rockfalls along faults, disruptions of slopes accompanied by strike-slip displacement, toppling of mountain peaks) are the results of both tectonic movements and the effect of gravity.

The seismo-gravitational phenomena (rockfalls, rockslides, ground avalanches and mudflows) occur within the isoseist range of VII° – VIII° MCS scale or more.

In his paper of 1977 Solonenko subdivided these three groups into seven types of seismogenic landslides and falls. According to Solonenko, the seismically induced landslides are distinguishable by their specific dynamics. The slid material is strewn over a much larger area than that of slides caused by gravity alone, particularly where there have been vibrations of long duration. Accordingly he explains the enormous extent (30 km^3 volume) of the Saidmarreh rockslide, Iran, where the rock debris travelled over a distance of 17 km and covered an area of 165 km^2.

Earthquake-triggered slope movements represent a serious economic problem in Japan. The Japanese islands are located in the Circum-Pacific seismic and volcanic zone, which accounts for the frequent occurrence of earthquakes inducing thousands of slope movements. All rock types are involved in the slides including metamorphic crystalline schists and phyllites, Tertiary coal-bearing sediments, mudstones, sandstones and tuffaceous rocks. The landslide problem is studied thoroughly and extensive remedial and control measures are carried out (Japan Society of Landslides 1972).

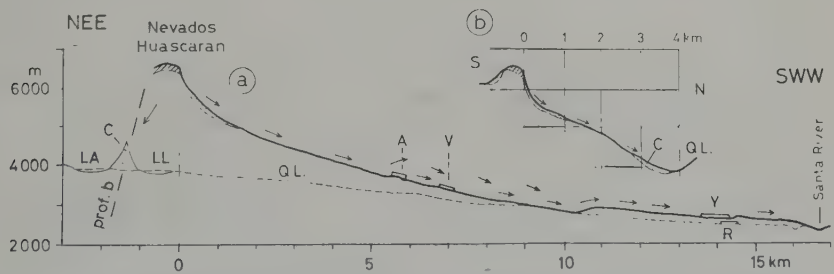


Fig. 2–9. Longitudinal profiles of the paths of the Nevados Huascarán rockslide which fell into the Santa Valley (a) and the Llanganuco canyon (b); QL — Quebrada Llanganuco, LA — Laguna Alta, LL — Llanganuco Lake, C — cone of debris. Destroyed villages Armapampa (A), Vuelta (V), Ranrahirca (R) and town Yungay (Y) (Mencl 1974).

Another seismically very active regions are the mountain ranges along the western coast of North and South America.

In 1960 an earthquake (magnitude 8.4 – 8.6) in Chile caused thousands of slides and rockfalls with disastrous results. In 1976 an earthquake in Guatemala (magnitude 7.5) triggered more than 10,000 landslides, predominantly in volcanic material. The distribution of landslides was found to depend on seismic intensity, and the lithological, morphological and tectonic conditions of the area. Many large landslides created debris-impounded lakes which threatened further damage as a result of the dam breaching (Radbruch-Hall and Varnes 1976).

In Peru one of the largest rockslides was caused by an earthquake in 1970 (magnitude 7.7), when the western part of the summit of Mount Nevados Huascaran collapsed. The displaced masses of volume 50 – 100 million m^3 buried a major part of the city of Yungay and several villages, killing 18,000 people (Plafker, Ericsen 1978). A similar avalanche was triggered by an earthquake in the same place in 1962. About 13 million m^3 of material were loosened and the city of Ranrahirca was buried. In 1970 a rock mass consisting of partly weathered biotite granodiorite fell from an altitude of 5400 – 6500 m onto the glacier. Snow and ice partly melted by the impact was incorporated together with morainic material into the debris, which formed a 16 km long debris flow moving at a rate estimated at 280 km, or in some

Fig. 2-10. Rockslide of Mount Nevados Huascarán, 1970. View of the NW part of the debris flow where it buried the town of Yungay (photograph by Novosad). 1 — Nevados Huascarán, 2 — site of the buried town of Yungay.

places up to 400 km per hour. At an abrupt break in the slope and at the morainic ridge the debris partly shot forth into the air to land approximately in the midst of the debris flow (Figs. 2–9, 2–10). The earthquake induced thousands of landslides within a radius of about 100 km around the city of Chimbote, near which was the position of the epicentre some 30 km off the coast of the Pacific Ocean.

In North America earthquake-triggered landslides occur most frequently in California; they range from small debris flows to slumps and rockslides of great size and depth. Slopes which are prone to sliding are formed predominantly of tectonically disturbed, poorly consolidated Tertiary sediments.

The Madison Canyon Rockslide, one of the major seismically induced rockslides occurred in 1959 in the Madison Canyon, Montana (U.S.A.), near the Yellowstone Park (Fig. 2–11). The earthquake (known as the Hebgen Lake earthquake of Mon-

Fig. 2-11. Madison Canyon rockslide triggered by an earthquake in 1959. 20 million m^3 of Precambrian rocks dammed the Madison River valley (Courtesy U.S. Geological Survey).

tana) caused additionally a number of minor landslides, slumps and earthflows, and longitudinal fissures opened along the mountain crests (Hadley 1959). The magnitude of the largest shock was 7.1 and the magnitudes of aftershocks which occurred during the following 24 hours ranged from 5.5 to 6.5. During the earthquake the crustal block containing the Hebgen reservoir subsided and several normal faults were reactivated. The subsidence in the area of the Madison Canyon rockslide was estimated at 2.5–3 m and it seems that this subsidence contributed to the slide movement.

The rockslide was set in motion right from the first and strongest shock; more than 20 million m^3 of Precambrian rocks, weathered gneiss, schists and dolomite slid onto the valley floor and climbed up the opposite wall within one minute. The slid mass was composed of angular blocks and debris of the aforementioned rocks, which formed spectacular bands deposited in the order in which they existed on the ridge. The most conspicuous of these was the white rampart of dolomite forming the highest part of the mass piled up against the northern slope (Fig. 2–12). According to Hadley (1978), the maximum speed of the slide was about 58 m s^{-1} in its western parts.

The rockslide blocked the Madison River forming a dam 60–120 m high which impounded a lake 60 m deep and 10 km long. Thanks to the timely excavation of

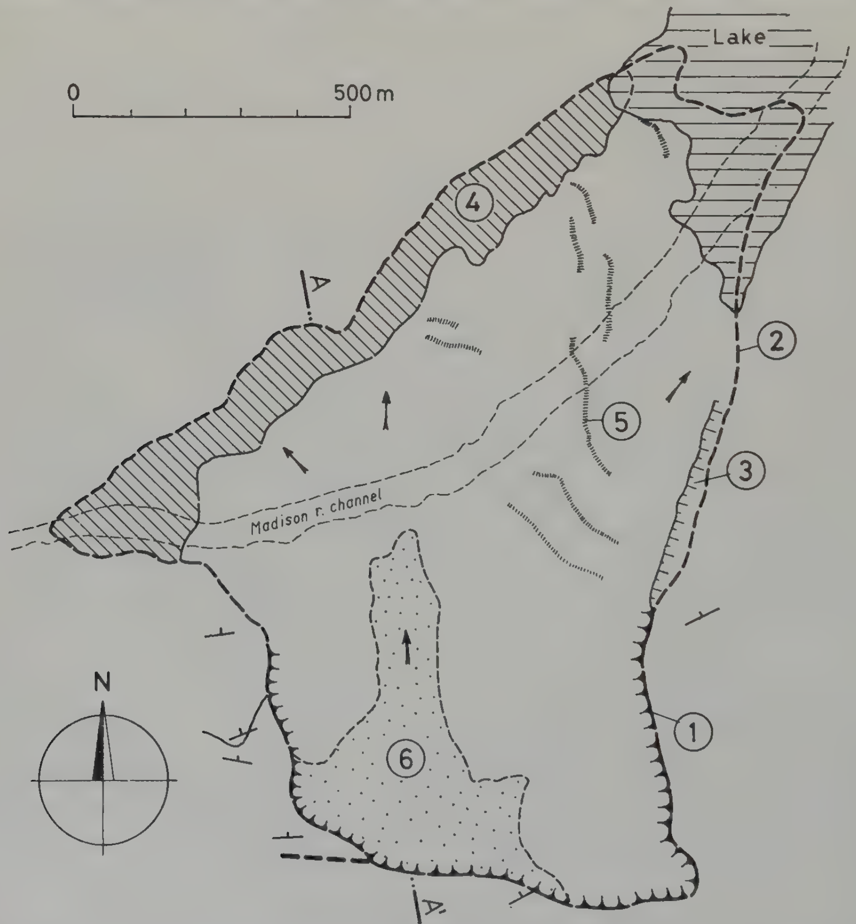


Fig. 2–12. Map of the Madison rockslide (after Hadley 1959); 1 — head area, 2 — outline of the slide, 3 — marginal ridge, 4 — dolomite debris, 5 — pressure ridge, 6 — afterfall.

a spillway channel in the natural dam, there was no sudden disastrous discharge of ponded water that would have endangered the downstream part of the valley (Harrison 1974). The Madison Canyon and the Hebgen Lake especially is a popular vacation area which at the time of the earthquake on 17 August 1959 was visited by many tourists, 28 of whom lost their lives during this disaster.

The controlling factors of the rockslide were the local geological and morphological conditions and the large input of kinetic energy from the earthquake. The canyon wall was very steep, mechanically unstable, and composed of fractured and weathered gneiss and schists maintained by relatively firm dolomite (Fig. 2–13).

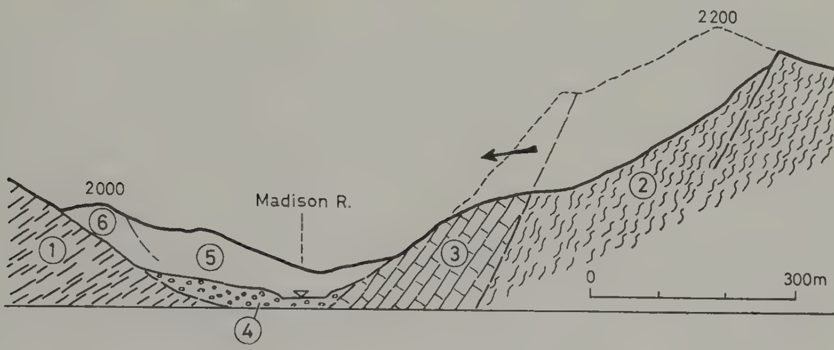


Fig. 2–13. Section of the Madison rockslide (after Hadley 1959); 1 — amphibolite and gneiss, 2 — gneiss and schist, 3 — dolomite, 4 — alluvium 5 — slipped material, 6 — dolomite debris.

In many rockslides the motion of the debris is known to be facilitated by an "*air cushion*". In the case of the Madison rockslide most of the air trapped beneath the slide escaped in one violent blast without contributing appreciably to the movement (Hadley 1978).

Another region of frequently occurring earthquakes with a high incidence of slope movements is Alaska. Since 1898 there have been 24 earthquakes of magnitude higher than 7. The earthquake of March 1964 was of magnitude 8.5 and triggered landslides over an area of 129,000 km^2. The main shock which lasted 1.5–7 minutes, was followed by about 12,000 aftershocks with magnitudes above 3.5 occurring within a period of 69 days. Seismically induced rockslides amounted to more than 2000, and the city of Anchorage suffered a great deal of destruction as a result of the interplay of seismic vibrations of long duration and the nature of the foundation ground. A delay of 1.5–2 minutes between main shock and the beginning of the mass movement indicated a progressive loss of strength of the sensitive silty clays (see Chapter 5, Fig. 5–69).

The Alaskan earthquake also triggered large submarine slides which caused additional damage to ports and on the coast; large sea waves destroyed piers, docks and many buildings.

In seismically active regions earthquakes are the predominant cause of slope failures. Seismically induced landslides occur in young tectonic mountain ranges and along active faults, particularly along the mobile marginal zones of continental blocks.

The chief factor in provoking slope movements is the intensity of the shocks; it has been established that earthquakes of magnitude 6.5 cause large slope failures in all landslide-prone areas. However, landslides and rockfalls may be triggered by shocks of lower intensity, particularly where the orientation of the horizontal components of the earthquake shocks is most conducive to motion. Also of importance is the duration of shocks; if the earthquake is prolonged, the vibration of rock debris and ascent of ground-water impair the stability of slopes. The degree of intensity that sets the slope in motion depends on a number of factors: geological structure of the slope, the properties of rocks, their disturbance by tectonic activity and weathering, the stage of morphological development of the area (i.e. the inclination and height of slopes), and the amount of precipitation in the period preceding the earthquake.

Many fossil rockslides and rockfalls in areas which are now seismically quiescent show the characteristics of earthquake-triggered slope movements. It is plausible that many instances of the gravitational spreading of steep mountain ridges and many block slides in Central Europe were induced by ancient earthquakes.

Chapter 3

THE CLASSIFICATION OF SLOPE MOVEMENTS

The great variety of slope movements offers much scope for different systems of landslide classification. Landslides can be classified on the basis of the mode and rate of the movement, the shape of the slide surface, the type of material involved and a number of other criteria.

Many authors have endeavoured to develop an appropriate classification of slope movements. Among these are Heim (1882), Howe (1909), Almagia (1910), Terzaghi (1925), Ladd (1935), Sharpe (1938), Emelyanova (1953) and Varnes (1958). From the engineering-geological point of view, Terzaghi's grouping of landslides based on the physical properties of the rocks involved has many advantages. Sharpe (1938) classified sliding movements according to the material displaced and the type and rate of movement, and studied the relationships between mass movements and geomorphological cycles and climatic factors.

In the Soviet literature, Savarenski's (1939) division into asequent, consequent and insequent landslides according to the shape of the slide surface is frequently used. Asequent landslides develop in homogeneous cohesive soils along curved, approximately cylindrical surfaces. Consequent landslides move on bedding planes, joints or planes of schistosity dipping downslope. Insequent landslides run transversely to bedding and are generally of large dimensions; the slide surfaces extend deep into the slope.

Popov (1951) has recommended that regional conditions be taken into account; owing to the diversity of controlling factors, regional conditions can further modify the individual landslide types.

In the American literature Varne's classification is generally used; its essential principles have been accepted by the Swiss Association for Rock Mechanics (1964). In 1978 Varnes revised and extended his classification establishing five principal types of mass movements and adding a sixth to accomodate landslides with the characteristics of several principal types:

I. Falls – The loosened rock mass is in free fall for the greater part of the distance of movement.

II. Topples – The rock mass overturns about a point below its centre of gravity.

III. Slides A. rotational – The rock mass moves about a point above its centre of gravity.

B. translational — The rock mass moves predominantly along more or less planar or gently undulating surfaces.

IV. Lateral spreads — Lateral extension movements occur in a fractured mass.

V. Flows A. in bedrock — These include spatially continuous deformations, and surficial and deep creep, involving extremely slow and generally non-accelerating differential movements among relatively intact units.

B. in soil — Movement occurs within a displaced mass, the form or apparent distribution of velocities of which resembles that of a viscous fluid.

VI. Complex slides — Landslides exhibiting a combination of two or more of the five principal types of movement listed above.

A classification based chiefly on mechanism of movement and morphology, with some emphasis laid on the rate of movement and the type of material involved has been proposed by Hutchinson (1968). He divided mass movements into three principal groups: I. creep, II. frozen ground phenomena, and III. landslides. Group I includes shallow soil and talus creep (1), deep-seated continuous creep (2) and progressive creep (3). The group of frozen ground phenomena encompasses solifluction, cambering, bulging and stone streams (4). Group III is subdivided into translational slides (5), rotational slips (6), falls (7), and subaqueous slides (8).

In Czechoslovakia, Nemčok, Pašek and Rybář (1972) have endeavoured to improve the classification and terminology of slope movements. What was proposed was grouping according to mechanism and rate of movement, with four basic categories: creeping, flowing, sliding and fall. Creeping involves a wide range of slow movements (a few centimetres per year) from talus creep to long-term gravitational deformations of mountain slopes. When the sliding mass contains so much water that the movement has the character of a flow, an earthflow or mudflow is involved. Relatively rapid movements along a definite sliding surface belong to the third category. Abrupt movements of solid rocks, in which free fall is a major feature, are placed in the fourth group.

The workable classification used in engineering geology should provide some general directions for investigation and the selection of preventive or remedial measures for each established group of slope movements. The authors believe it is most convenient to classify sliding phenomena according to regional conditions. Since the major part of all landslides in Czechoslovakia involve Quaternary cover deposits, these are placed in a separate group. Slope movements in the pre-Quaternary bedrock are divided according to the character of the rocks and the type of movement.

Using these principles the following scheme has been proposed (Záruba, Mencl 1969):

A. Slope movements of superficial deposits (slope detritus, weathering material), produced mainly by subaerial agents:

(a) Talus creep which also causes the terminal bending of strata.

(b) Sheet slides.

(c) Earth flows.

(d) Debris flows, muren, liquefaction of sand.

B. Slides in pelitic rocks (clays, marls, claystones, clayey shales, etc.):

(a) Along cylindrical surfaces.

(b) Along composite sliding surfaces.

(c) Caused by the squeezing out of soft underlying rocks.

C. Slope movements involving solid rocks:

(a) Rockslides on predisposed surfaces (bedding, schistosity, jointing or fault planes).

(b) Long-term deformations of mountain slopes.

(c) Rockfalls.

D. Special kinds of slope movements which constitute important geological phenomena in particular countries:

(a) Solifluction.

(b) Slides in sensitive clays (quick-clays).

(c) Subaqueous slides.

Individual types of slope movement are characterized in detail in Chapter 5, which also gives examples and case histories.

Fig. 3–1. Pines shifted and bent as a result of sliding at the foot of Mužský Hill in 1926 (photograph by Záruba 1963).

3.1 Geologico-morphological development of landslides

Other parameters that serve as a basis for a broad division of slope movements are geologico-morphological development and age. The formation and history of slope movements are both greatly influenced by the time factor. Since some agents change with time, every mass movement undergoes a gradual development. At first, indications of the disturbance of equilibrium will appear, i.e. fissures form in the upper part of the slope. Later the loosened mass begins to move, travels down the slope and gradually accumulates at the foot of the slope. According to the stage of development we may speak of the initial, advanced and final stages of slope movements.

On the basis of age, *contemporary* (recent) and *ancient* landslides are distinguished. Ancient landslides that cannot be reactivated under the present climatic and morphological conditions are termed fossil landslides. Where they have become covered by loess loam or other deposits they are referred to as *buried landslides.*

Fig. 3-2. Road dislocated along a crack suggests recent movement (photograph by Záruba).

For engineering purposes it has proved convenient to distinguish between active, dormant (potential) and stabilized landslides according to the degree of stabilization.

Active landslides are easy to recognize from their configuration, because their surface forms are fresh, conspicuous, and as yet not effaced by rainwash and erosion. Trees are diverted from their original position (Fig. 3 – 1), roads and tracks traversing the affected area are interrupted (Fig. 3 – 2) and buildings are often damaged.

Dormant (*potential*) slides are usually overgrown or obscured by erosion, so that traces of the last movement are difficult to discern. Since the slide-provoking factors are still operating, the movement may recur.

Stabilized landslides which were initiated under morphological and climatic conditions that cannot occur today are permanently at rest.

Chapter 4

MECHANICS OF THE DEVELOPMENT OF SLOPE FAILURES

4.1 General aspects

The aim of this chapter is to discuss the stress and deformation states that are either present or under development within slopes; procedures for analysing the stability of slopes are dealt with in Chapter 7. Because of the advanced methods of analysis involved, this topic has been the subject of several treatises. Thus the classical stability computations have been refined, but also other criteria have been introduced for the evaluation of slope safety. Let it be admitted that the present approach to refinement of the classical methods tends to confirm that these methods yield low safety factors, and therefore several improved procedures giving higher values have been drawn up. On the other hand, it has been demonstrated how much the progressive development of slope failures reduces the factors of safety obtained by the classical procedures. Thus there is undoubtedly a need for a more direct approach and the authors hope to make some contribution to a better understanding of the static conditions present in slopes. Hand in hand with this, some attention must be given to specific prevention measures and the improvement of their effectiveness.

Several ways are possible in order to analyse the problems as outlined. We have chosen one which seems to be promising and is reflected by several questions which appear in the discussions and which can be summed up in the simple question: In which part of the slope does inception of the failure take place? To answer this, a distinction has to be made between the often unseen yield of displaced material and the extent of the phenomenon visible at the surface. Although both of these quantities will be considered in the following paragraphs, the former promises to be more useful and therefore is the main focus of attention of this chapter.

These problems of slope safety are, however, greatly affected by the non-homogeneity and anisotropy of the earthen mass as well as by conditions in the environment. Consequently, the validity of the results of analyses depends on the ability of the investigator to give a true picture of the geological properties of the site in relation to the mechanical properties of rocks. The initial stresses present in the ground are also important, as well as the hydrogeological conditions, the intensity of the influence of ground-water and frost or other climatic forces which attack the slope. The classical stability analyses deal with less complex conditions. This is particularly true of the simplified methods and diagrammatic aids that designers often use. The latter in particular deal with slopes as if they consist of homogeneous and isotropic material.

Therefore the simpler conditions will be considered first and those that are geologically more complicated will follow. Clayey soils, owing their sensitivity to physical conditions, provide the best starting point.

4.2 Failures arising at the toe of the slope

The majority of slope collapses spread from the toe. It should be noted that the local factor of safety varies from point to point within a slope. Generally the danger of shear failure is much greater near the toe than higher up the slope. Fig. 4–1 shows the isolines of the local factors of safety in a slope in stiff clay during the three stages of excavation of a cutting, to depths of 10, 13.5, and 16 m, respectively. The deformation modulus for the clay on unloading, 51 MPa, has been assumed to vary with both compression and shear stresses. Bulk shear strength parameters $c' = 0.025$ MPa, $\varphi' = 18°$, with small decrease near the coordinate origin. Dilatancy has been introduced into the analysis. Although the general factors of safety (obtained by the classical methods of analysis) are of the order of 3, 2.6, and 2,1, respectively, it can be seen how the local factors of safety at the toe differ from those at the top of the slope, right from the beginning of the excavation. This provides evidence of the inception of progressive failure despite the relatively high general factor of safety.

Note. Three different denotations are used in this book for the factor of safety:

The *local factor of safety*, which indicates the ratio of the strength (mostly shear strength) of the material to the corresponding stress magnitude present at the individual points of the studied body. It can be exactly determined only by using the modern mathematical tools of static analysis.

The *general factor of safety*, which indicates the ratio of the sum of the total forces which the material of slope is capable to mobilize due to its strength along a slip surface, to the sum of actual forces tending to slip the slope. The magnitudes of all the quantities are calculated by using the classical methods of static analysis (Chapter 7).

The *overall factor of safety* indicating the same ratio as the general factor of safety, but with the distinction that the magnitudes of the quantities are calculated by using the modern mathematical methods of analysis.

The static condition of the slope as described above is connected with the differences in displacements that occurred after the slope was modelled. In Fig. 4–2 the horizontal displacements of the slope shown in Fig. 4–1 (c) are plotted. Horizontal displacements have been preferred to the total displacements because the latter include not only the shear deformations of the slope but also the overall upheaval caused by the excavation of the cutting. Therefore horizontal displacements better reflect the shear deformations. Fig. 4–2 indicates that the greatest horizontal displacements (6.7 cm) are not near the toe but are inside the slope mid-way up. The toe of the slope, supported by the bottom of the cutting, tends to oppose the displacements and therefore

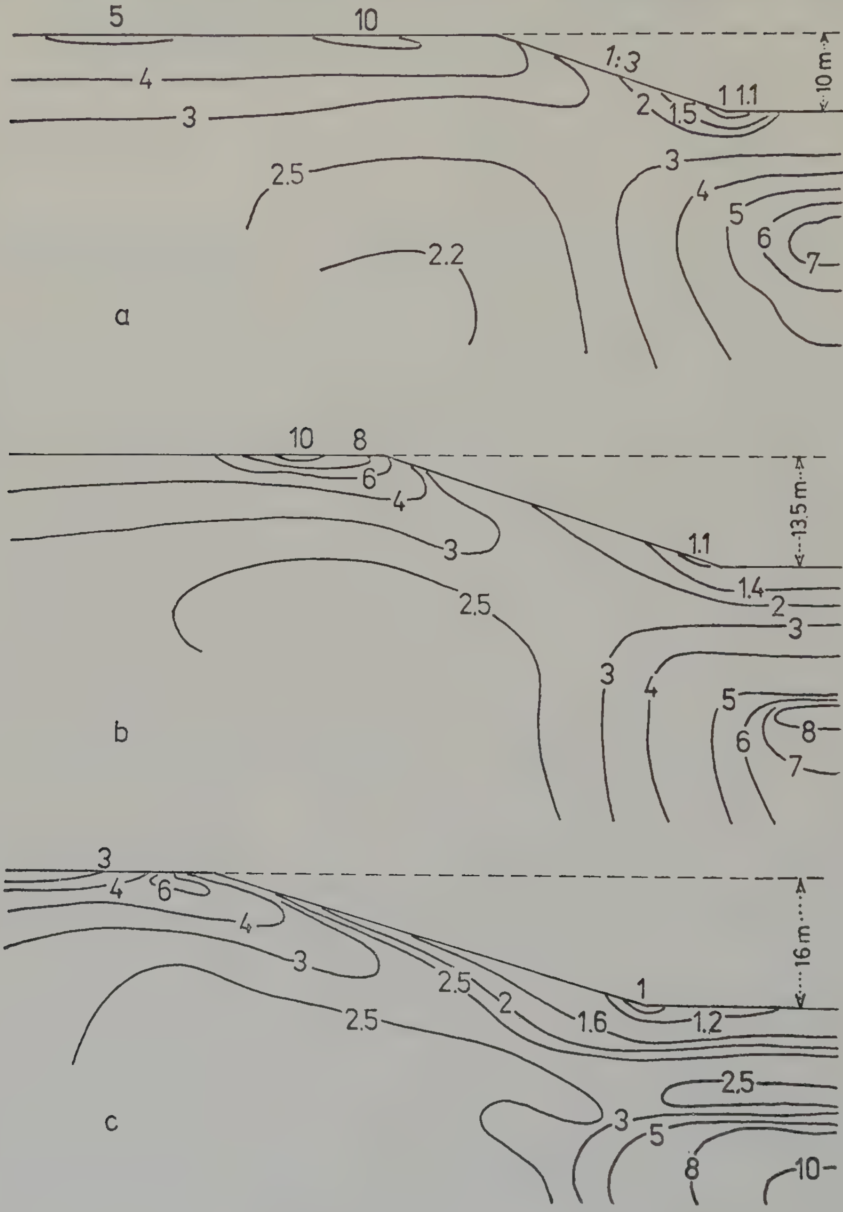


Fig. 4–1. Computed values of the factors of safety in slopes occurring in (a) 10 m deep, (b) 13.5 m deep, and (c) 16 m deep cuttings excavated in stiff clay. Initial horizontal stress $K_0 = 0.75$. Groundwater is assumed to be absent. The small values of the factors of safety at depth generally result from the assumed low magnitudes of the initial horizontal stress (Mejzlík and Mencl 1975).

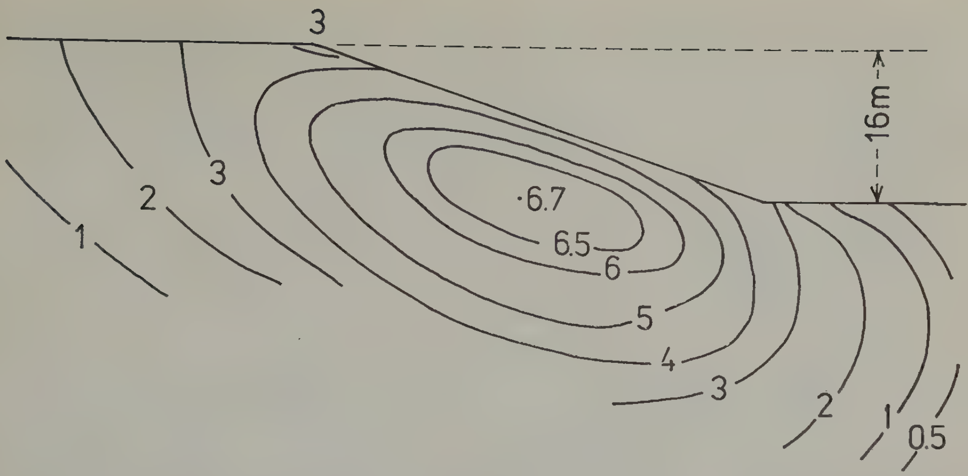


Fig. 4–2. Horizontal displacements (cm) of the slope shown in Fig. 4-1(c), the greatest displacement being within the slope at mid-height.

is subjected to relatively high stresses. This is also illustrated in Fig. 4–3, which gives diagrams of the normal and shear stresses acting at different vertical cross-sections of the clay mass above a potential slip surface. From the stress diagrams the normal and shear forces *E* and *X* and their resultants *R* have been plotted. They demonstrate the tendency of the entire clay body to bear against its toe, in spite of

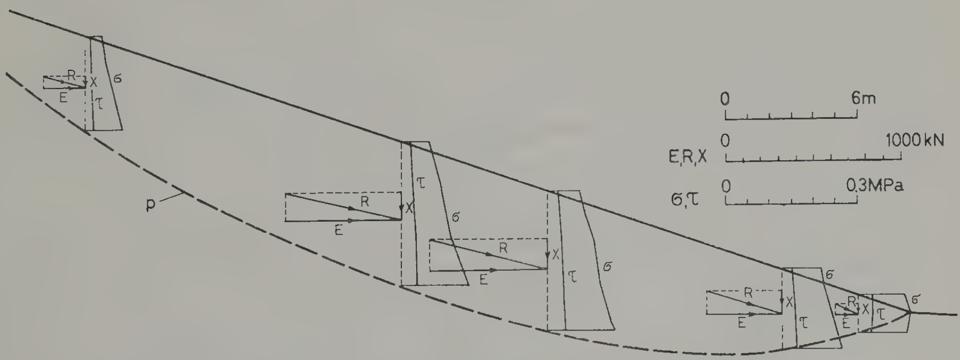


Fig. 4–3. Normal and tangential stresses (MPa), corresponding normal and tangential forces *E* and *X* (kN) and their resultants *R*, acting at different vertical cross-section positions of the slope shown in Fig. 4—1 (c); p — potential slip surface.

the fact that the general factor of safety is of the order of 2.1, as indicated above. The resultants are almost parallel to the slope surface.

The measurement of relatively large distortions in clay near the toe of a slope in a canal cutting at La Fléchère (Belgium) was referred to by de Beere (1969). The cutting is 16 m deep and its lowest 6 m were excavated in Eocene beds (Ypresian clay), Fig. 4—4. The distortions recorded by inclinometer *a* at an elevation of 114.5 m were several times greater than those recorded by inclinometer *b* at an elevation of 115.4 m. The latter also began to develop 11 days later than the former. The clay content of the soil was 19—23% and several slope failures occurred during dredging. Bored piles in two rows were put in to stabilize the slope toe.

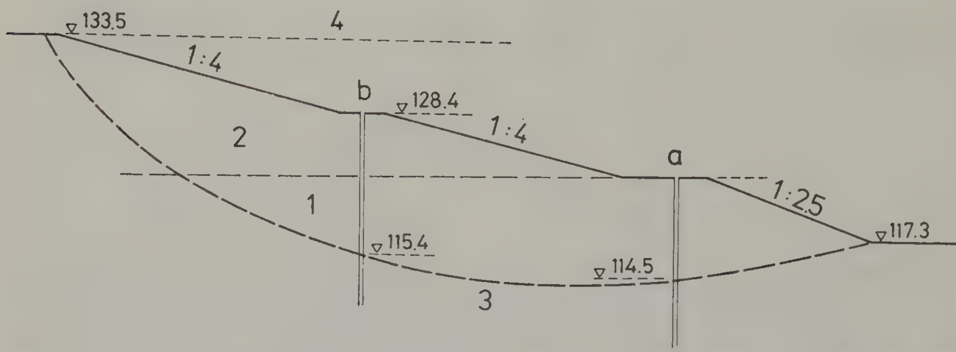


Fig. 4–4. Cross-section of the canal cutting at la Fléchère (de Beer, 1969); 1 — Ypresian clay, Eocene, 2 — loam, a, b — inclinometers, 3 — developing slip surface, 4 — original ground surface.

Under the climatic conditions of Central Europe the likelihood of the yielding of the soil in the toe is increased by frost action. The resulting high water content weakens the clay and decreases its resistance.

Another crucial factor is the presence of ground-water which reduces the normal stresses and increases the shear stresses along a potential slip surface. Fig. 4—5 illustrates the horizontal displacements in the same slope as that shown in Fig.

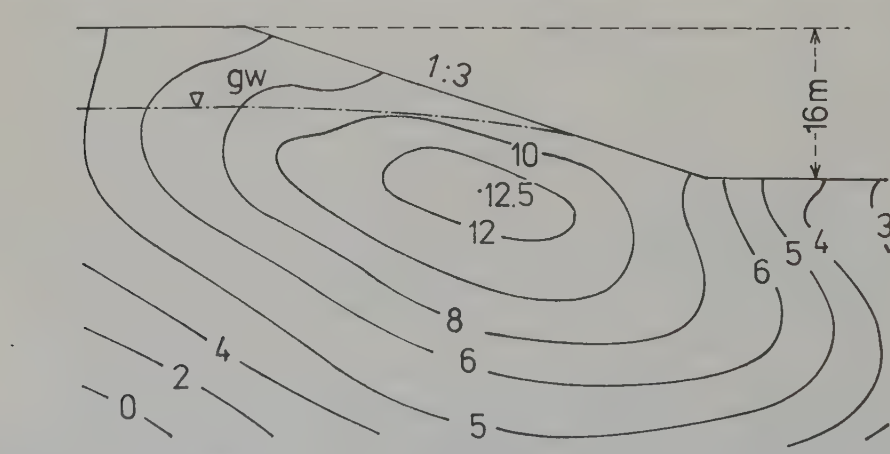


Fig. 4–5. Horizontal displacements (cm) in the slope shown in Fig. 4—2 in the presence of ground-water (Mejzlík and Mencl 1975).

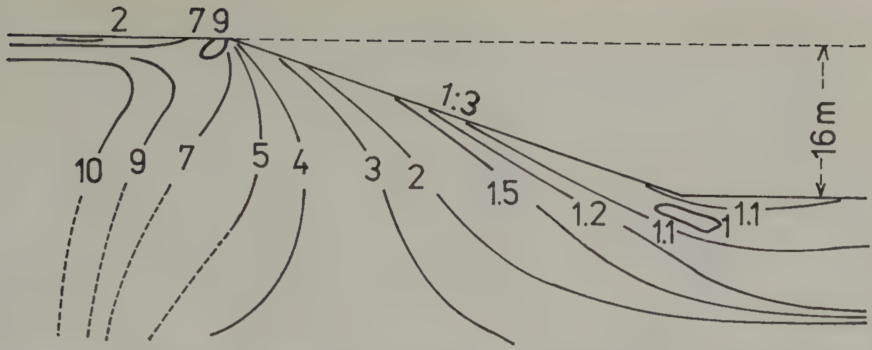


Fig. 4-6. Computed values of the factors of safety of the slope shown in Fig. 4-1(c), but with a coefficient of initial horizontal stress $K_0 = 1$. Dotted sections of the curves indicate that the stress field has been influenced by the boundary conditions of FEM analysis.

4–2 in the presence of ground-water with the original level at a position about half way up the slope. The increases in the horizontal displacements are striking as are the differences between the displacements near the toe and those in the lower third of the slope surface. This deformation is already visible on the slope face, although cracks have not appeared in its upper part.

The presence of a considerable initial horizontal stress in the ground makes the situation more serious. Fig. 4–6 shows the lines of the factors of safety in a slope similar to that in Fig. 4–1 (c), but taking $K_0 = 1$ as the coefficient of horizontal stresses. The safety factors in the lower part of the slope are smaller and the lowest values, instead of being near the surface, occur within the soil mass beneath the toe. This coincides with the seat of maximum tangential stress which reaches a value of 0.097 MPa (Fig. 4–7a). The overall factor of safety decreases to less than 1.6 (a magnitude of 2.1 again being valid for the general factor). The difference is caused by the existence of great horizontal forces (P in Fig. 4–7b), which do not enter into the classical analyses. The potential slip surface occurs at a lower position than that obtained at $K_0 = 0.75$.

These tendencies appear much more pronounced when analysing slopes with a value of K_0 greater than one. New features develop which change the nature of the behaviour of the slope, as will be discussed in the following section.

4.3 Failures arising within the slope

Several types of failure of high slopes give rise to a distinct, polished outcropping of the slip surface at the top. Surprisingly, when excavating into the slipped mass, the continuation of the slip surface cannot be found within the lower part of the slope. Nevertheless, the origin of the slope deformation is to be found in this lower part.

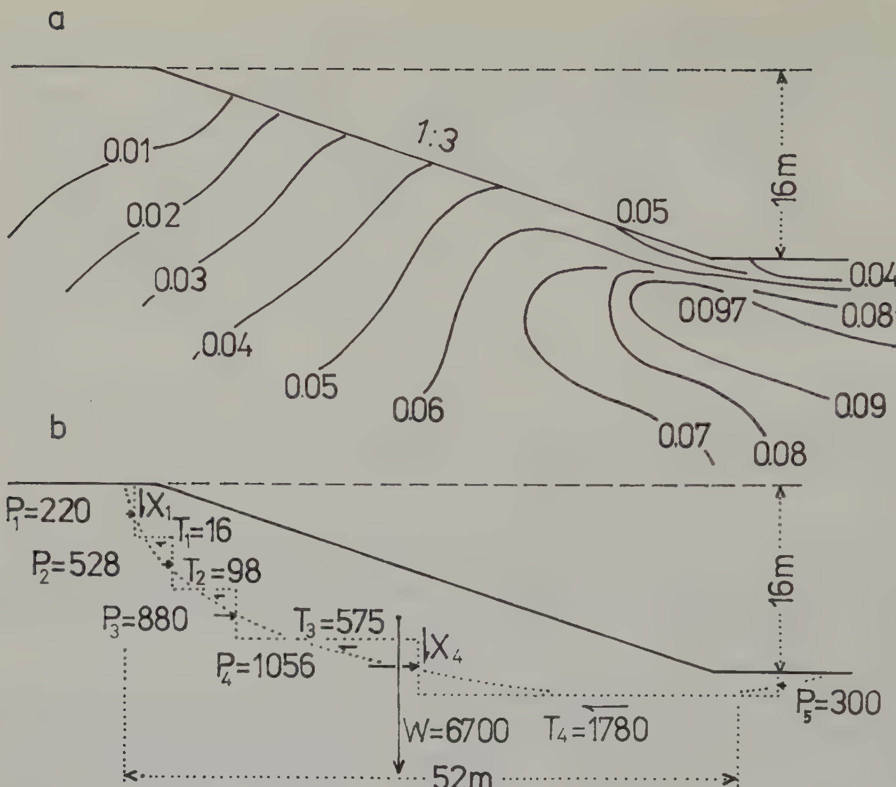


Fig. 4-7. a — Lines of maximum shear stresses (MPa) of the slope shown in Fig. 4—6; b — great horizontal forces *P* (kN) acting on the potential sliding surface from within the slope mass are opposed by shear forces *T*.

In order to understand this phenomenon, it is necessary to consider the two modes of shear failure that occur in soil and rock masses. It is not intended to discuss the problem in full here, as the subject has already been covered adequately (Subsections 3.4.4 and 3.4.5, Záruba and Mencl, 1976). Only let it be recalled that the shear failure of the soil or rock structure begins when the shear force reaches 60 per cent of the shear strength and manifests itself by either of two secondary but nevertheless important phenomena: (1) by an increase in volume due to shear strain (referred to as a strain-softening or dilatancy), with a brittle failure and the development of a thin slip surface, or (2) by a decrease in volume (referred to as negative dilatancy or contractancy) leading to a ductile failure and the development of a thick shear zone. The former mode occurs under conditions of relatively small pressure in the mass, whereas the latter takes place when the mass is subjected to high pressures. Thus, e. g., the delimiting pressure between the two modes of failure is of the order of about 0.4 MPa for stiff clay and about 0.8 MPa for claystone.

Fig. 4–8 illustrates diagrammatically the presence of both modes of failure in a slope, and Fig. 5–60 shows an actual case of this.

Fig. 4-8. Development of brittle shear failure B (a thin slip surface) and ductile shear failure D (a thick shear zone) in a slope (Mencl 1968).

The development of the slope movement as influenced by the ductile yielding of the mass is again progressive, this time in an exaggerated mode. On account of its thickness, the deep ductile shear zone produces large displacements even when the overall factor of safety is of the order of 1.6. The boundary zones of the deformed mass (inclined to develop brittle shear failure) are carried along with the displacements of the shear zone. This leads to a slip failure in the boundary section of the potential slip surface although the overall factor of safety is distinctly greater than one.

The relatively large displacement of a rock mass under conditions of ductile behaviour can be studied from the results of field block shear tests (Fig. 6–21).

As can be seen, the presence of two factors is necessary for ductile behaviour: a compressive stress over a certain limit, and a factor of safety in shear less than about 1.6. High compressive stresses may be generated by two stress fields:

(a) The weight of the material. Fig. 4–9 shows the principal stresses acting in the vicinity of the slope referred to in Fig. 4–7. An examination of Figs. 4–7 and 4–9 indicates that the two factors mentioned above do not exist together in this slope. Probably the depth of the cutting would need to be about 22 m to obtain the expected mode of failure under specific conditions.

As the volume of the soil in the ductile shear zone decreases, the pore water pressure increases and the stability of the slope becomes less than anticipated. Such a development proceeds rapidly so that, when the stability of the slope decreases to unity, the failure of the slope occurs soon after the excavation has been carried out. In contrast, the slopes mentioned in section 4.2 exhibit a delayed type of failure because the volume increase along the slip surface decreases temporarily the pore water pressure. In slopes which are relatively stable, because they exhibit the overall factor of safety

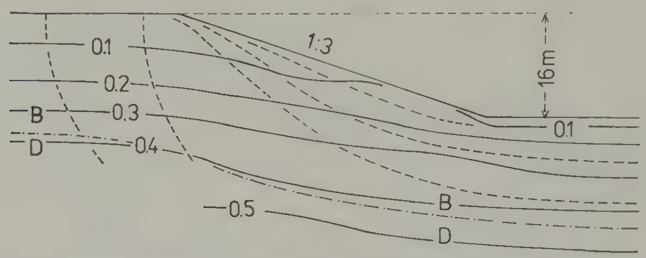


Fig. 4-9. Major compressive stresses (MPa, unbroken lines), their directions (broken lines) and the boundary (dot-and-dash line) between the areas of potential brittle (B) and ductile (D) failures in a slope referred to in Fig. 4—7 (Mejzlík and Mencl 1975).

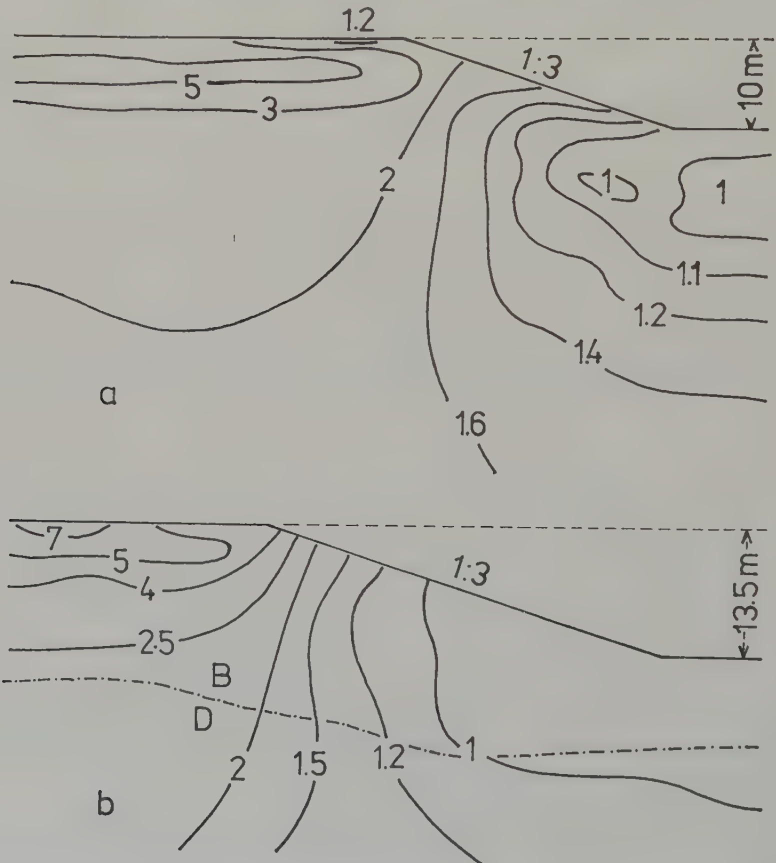


Fig. 4-10. Computed values of the factors of safety for the slopes illustrated in Fig. 4—1(a) and (b), but taking the coefficient of horizontal stress as $K_0 = 1.5$; B-D — boundary between the areas of potential brittle and ductile shear strains.

greater than unity, creep phenomena (very slow downhill deformations) are intensified by ductile strains. For these reasons the behaviour of a slope with a deep ductile shear zone differs considerably from that described in section 4.2. In this way an important classification criterion presents itself — one that may not be recognized even by experts in the subject — namely the distinction between low slopes and high slopes, the latter being distinguished by a much greater tendency to ductile yielding.

Several large man-made landslides, e. g. those of the slopes of deep opencast coal mines, belong to this category. Also the creep movements of mountain slopes (section 5.3.2) are essentially of the same character. The slide movements of the slopes of the Panama Canal (Legget, 1962) developed along weak shale layers within sandstone beds (Lutton et al. in Rockslides and Avalanches, B. Voight Ed., 1979). This was the reason why ductile yielding did not develop at a large scale, although the depth of the slip surface amounted to 90 m.

(b) The tendency towards a ductile mode of sliding is increased by large horizontal stresses in the ground. This can be illustrated by the analysis of the static conditions in the slope shown in Figs. 4–1 (c) and 4–6, but taking a value of $K_0 = 1.5$ for the coefficient of horizontal stress. Fig. 4–10 (a) and (b) show the lines of equal factors of safety for two depths (10 m and 13.5 m) of cutting. Excavation to a depth of 16 m cannot be achieved because a slope of that height in not stable.

As can be seen, the zones of local factors of safety less than one spread out and reach to greater depths. At the same time the maximum tangential stress exceeds 0.17 MPa. Line B–D in Figs. 4–10 and 4–11 separates zones of brittle and ductile failures. A considerable section of slip surface develops, although the general factor of safety according to the classical approach is of the order of 2.6, as noted earlier. Taking the section of failure of the slip surface as well as the ductile yielding of the shear zone into consideration, the overall factor of safety of the order of 1.4 is obtained for the given slope of 13.5 m height. This illustrates the importance of the consideration of the progressive failure.

It should be emphasized that although the factors of safety are between 2 and 5 in the upper section of the potential slip surface, the lower section of the slip surface has already developed. Its outcrop may be visible on the slope surface (Mencl et al., 1965). Outcrops tend to open in dry weather so that subsequent rain deteriorates the slope.

The progressive nature of the slope movement is very evident and highlights the difficulty of treating the slope analysis as a simple stability problem. In addition, a new and somewhat unexpected phenomenon appears: in the zone denoted by U in Fig. 4–11 the sum of the principal stresses increased after the cutting had been excavated. As a result pore water pressure increased, resulting in decreased effective stresses and therefore in a lower factor of safety also.

The zone of ductile deformations and small factors of safety includes also a region of several metres depth below the cutting bottom. Therefore there is a ductile yielding and an additional upheaval of the bottom (i. e. additional to that produced by the removal of load during excavation), but it does not lead to a general failure of the slope. The more dangerous slip surface has its seat higher up as shown in Fig. 4–11.

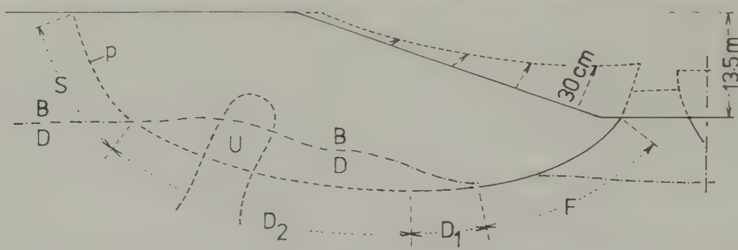


Fig. 4–11. Progressive development of the slip surface in a stiff clay slope (as in Fig. 4–10b). Initial stress ratio $K_0 = 1.5$; p — potential slip surface; B-D — boundary between the areas of potential brittle (dilatant), and ductile (contractant) shear strains; D_1 — section of the slip surface along which the ductile shear strain can develop, the local factors of safety being < 1.5, and the section being situated in area D; F — sections of the slip surface along which the factors of safety are < 1 and therefore $\varphi = \varphi_r$; U — zone in which the value of $\sigma_1 + \sigma_2 + \sigma_3$ (compression) becomes greater than the original value; D_2, S — sections of the slip surface in which $F > 1.5$ (Mejzlík and Mencl 1975).

Development of the slip zone to an even greater depth occurs when still greater initial horizontal stress is present in the ground. Fig. 4–12 shows an example of a railway cutting excavated in the deposits of the Palaeogene of the Inner Carpathians. Horizontal compression of the order of 0.4 to 0.6 MPa was measured at the bottom of the stable part of the 16 m deep cutting, using the technique described in section 6.1.9. A comparison of the latter values with those indicated in Figs. 4–9 and 4–13 (below the bottom of the cutting) seems to indicate that the coefficient K_0 might have been greater than 1.5. The bottom of the excavation was heaved up about 3 m and the kneaded character of the squeezed mass could easily be observed in the excavation for the supporting invert frames.

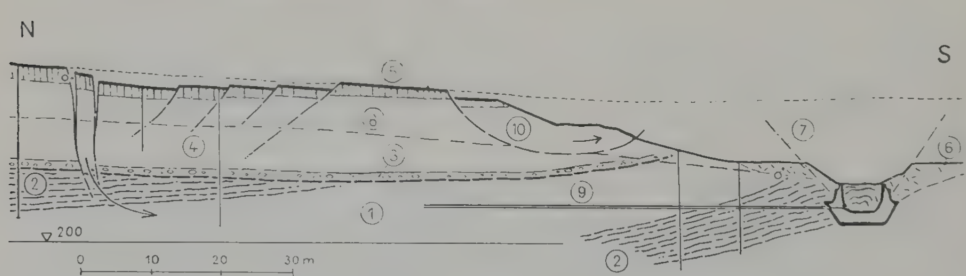


Fig. 4–12. Cross-section of the landslide in the railway cutting in Bánovce (Slovakia); 1 — Palaeogene sandstones and argillaceous shales, 2 — Palaeogene claystone, 3 — Neogene clayey gravel, 4 — Neogene clayey silt, 5 — loam, 6 — fill, 7 — limits of the wedge squeezed up into the cutting, 8 — ground-water table (after slope failure) in Palaeogene sandstone, 9 — horizontal drainage borings, 10 — slip movement at the top of the slope, which occurred prior to total collapse of the cutting.

4.4 Failures occurring on the face of the slope at about its mid-height

If geological conditions are uniform, this type of failure mostly occurs when ground-water is present. Rather than go into a less convincing theoretical analysis, let us take the instructive example of a railway cutting in the Neogene basin of East Bohemia (Fig. 4–14).

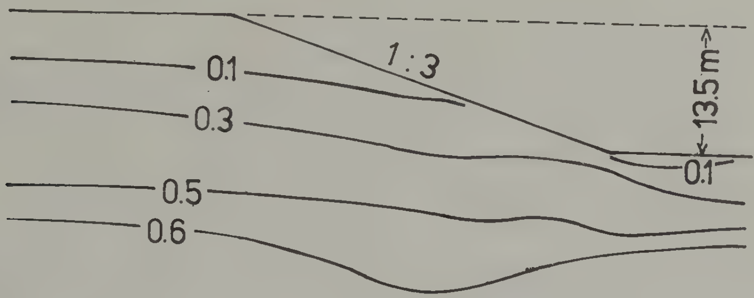


Fig. 4–13. Lines of equal compressive principal stresses (MPa) in the slope illustrated in Fig. 4–10b.

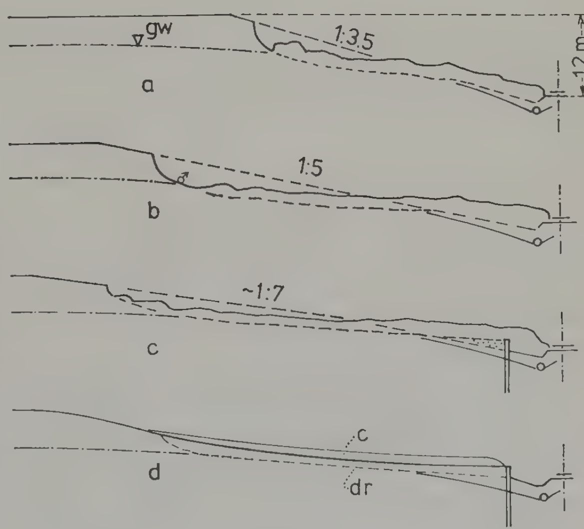


Fig. 4–14. Four stages of the development of excavations of a railway cutting near Česká Třebová, eastern Bohemia; (a) original excavation and failure of the slope, (b) first reconstruction and second failure, (c) second reconstruction using a pile wall and third failure, (d) final state; gw — ground-water table, dr — gravel-filled drain trenches 8 m apart, c — clay cover of trenches.

Because confidence was placed in the classical theory of slope stability, the slope was first designed as shown in Fig. (a). The parameters of the strength of undisturbed, fissured stiff clay were assumed to be of the order of $c' = 0.018$ MPa and $\varphi' = 17°$. The lowering of the ground-water table by the drain at the toe was expected to provide adequate protection for the slope. Nevertheless the slope collapsed progressively due to frost action, the failure spreading from half way up where the water table was nearest to the slope face. The slope was reconstructed twice (Fig. b and c), with the eventual use of a pile wall. However, the third version (Fig. c) of the slope failed after three years, on account of the ground-water table being near the slope face at mid-height. Softening of the clay by frost was distinctly evident in spring, and only rare species of plant become established in the middle region of the slope, although generally the slope was thickly overgrown. In order to remedy the disastrous effects of frost action on the slope surface, the latter was drained by flat drains 8 m apart (Fig. d). The excavated material from each drain was moved with a bulldozer over the previously excavated and filled up drain thus forming a cover against frost penetration to the drain. This arrangement has proved to be effective. It may be noted that failure did not occur in those sections of the slope where trial horizontal drainage boreholes had been made. The case illustrates the delicacy of trying to avoid the risk of slope failure by designing low angle slopes.

The source of water causing this type of slope failure may also be fed by precipitation as noted by Terzaghi (1936). Fig. 4–15 illustrates two stages of failure development in a slope comprising Miocene silty clay. During the period of excavations

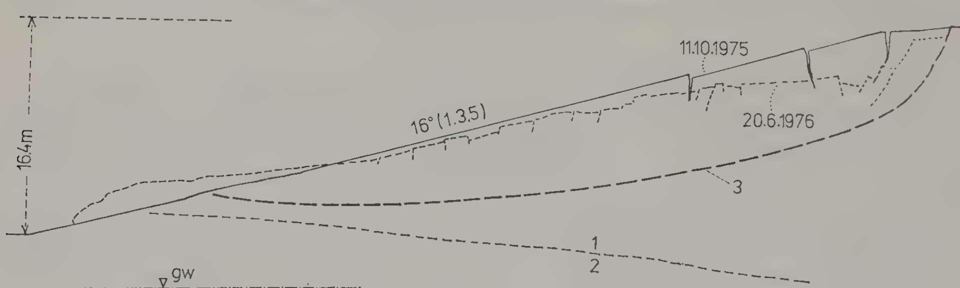


Fig. 4–15. Development of slope failures in a 16 m deep railway cutting in Miocene silty clay (water content 25 to 28 per cent, liquid limit 55, plastic limit 20) in southern Slovakia; 1 — silty clay, 2 — clayey sand, 3 — slip surface, gw — ground-water table (Slivovský, personal communication).

the ground-water level was undoubtedly very low, so that the increasing amount of water in the clay must have been derived from the surface. The more the clay is fissured the more likely it is that surface water can penetrate it.

The penetration of water from the surface is probably aided by the extension of the ground surface behind the top of the slope caused by the excavation. Referring to the slope illustrated in Fig. 4–5, this extension is of the order of 0.16% in the horizontal direction. For the slopes of 10 m and 13.5 m height the extension is of the order of 0.05% and 0.09%, respectively. When extrapolated, these values indicate that an extension in the horizontal direction of 1% can be expected near a cutting of about 20 m depth. An extension of 1% at the surface corresponds to an increase in porosity of about 2.5%, which produces a decrease in shear strength from about 0.08 MPa to about 0.05 MPa in a saturated stiff clay. Or, when the surface is formed by a hard layer – a situation which occurs often in Neogene clay – fissures some centimetres wide develop.

4.5 Failures originating at the top of the slope

Almost all collapses caused by the imposition of greater loads on the slope surface belong to this category. Loading may result either from embankment fills or from the slope masses displaced from above. The latter occurs with the gravity changes arising from earthflows and sheet slides. Shear forces increase in the slope soil with the additional loading, yet the shear resistance cannot increase because of the slow consolidation process in the saturated clay.

The situation is, however, more intricate under field conditions. The process of consolidation decreases the water content of the clay underneath the increased load, and shear strength begins to build up. On the other hand, the pressure of the escaped water decreases the shear strength in the surroundings and the overall result is an adverse effect on the stability. Analysis of this problem will be dealt with in section 11.4.

Failures originating at the top of the slope often have more than local impact. They indicate a loosening of the top layers (as already noted at the end of section 4.4.) to such a degree that some considerable movement towards the cutting can be expected. An example is shown in Fig. 4–12, and the authors have observed the phenomenon in several other cases. Thus a simple slip movement at the top may signal extensive destruction of the entire slope.

4.6 Complex modes of failure

The straightforward developments of slope movements as described in the foregoing become more intricate under more complex geological and topographic conditions. Although the preceding analyses remain valid, further variations of the phenomenon occur which affect the mode of failure.

One of the more important of these occurs when brittle beds under a slope overlie more plastic beds. Support at the toe is absent and this reduces the horizontal compression in the overlying beds and diminishes the factor of safety in the upper part of the slope. Fig. 4–16 shows the results of finite element analysis of a Neogene slope in Košice, East Slovakia. The shaded areas represent silty clays varying from plastic to stiff; non-shaded areas indicate silty to clayey gravels of relatively low density. This can easily be explained by the slope history which is characterized by the lines of equal safety factors. Because of yielding of clay layers the horizontal stresses in the gravel decreased, and this is reflected in the magnitudes of the safety factors. Although the general safety factor along the potential slip surface drawn in Fig. 4—16 is of the order of 1.9, the local safety factors are low, even as low as one, in the upper section of the slope. It is noteworthy that the low safety factor values occur in gravel areas, while those in clay are considerably greater. This is characteristic of this mode of slope yielding.

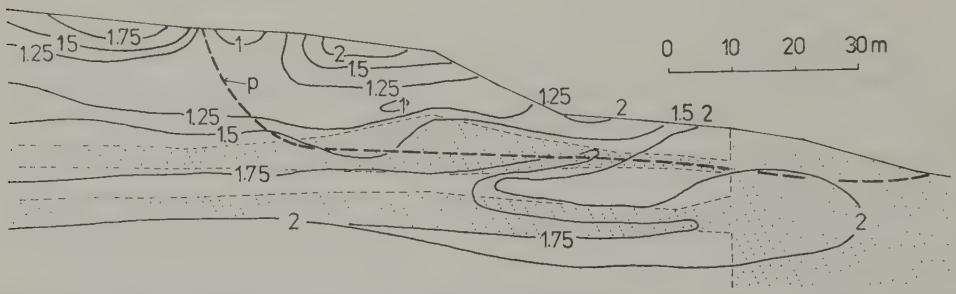


Fig. 4-16. Static conditions in the profile 10/IV of a slope in Košice, eastern Slovakia. Shaded areas — Neogene plastic to stiff silty clay; unshaded areas — Neogene silty and clayey gravel of medium density; unbroken lines — lines of equal safety factors; dashed line-the least safe of three potential slip lines (p), analysed by the finite element method (Mejzlík and Mencl 1975).

The decreased compression of the top gravel layer is illustrated in Fig. 4—17, which shows the isobars of the horizontal compression stress in the same slope. In the neighbourhood of area *A* the compression has decreased, and has even reversed to produce tension near the ground surface. It is of interest to observe how the compression starts to increase as the thinning section of the upper silty clay bed is ap-

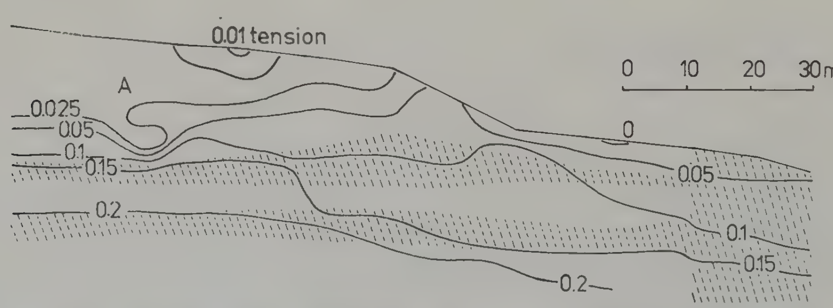


Fig. 4-17. Lines of equal horizontal compression stress (MPa) in the slope shown in Fig. 4—16; the decrease in area A is caused by bulging.

proached, and then to decrease again near the less resistant infilling forming the bottom of the valley. The reduced compression in the top gravel layers lowers the overall stability of the slope. In any event the magnitude of the friction angle in the gravel is affected, because it decreases with the loosening of the gravel structure. This, of course, can be recognized during the investigation of the site, and the tensile stress can be measured as was done in the case shown in Fig. 4 – 17 using the technique described in section 6.1.9.

Higher slopes exhibit *ductile yielding* at depth because the rock undergoes ductile behaviour when subjected to high compression (section 4.3). From the mechanical point of view the two types of behaviour of a rock – ductile at the depth and brittle near the surface can be considered as if two layers of rock of different mechanical properties existed. As we have noted in section 4.3, ductile yielding is accompanied by pronounced creep deformation which diminishes the stability of the slope. Therefore relatively large movements of the slopes are exhibited by small surface gradients. This probably may explain the very gentle dips of several nappe structures, with which the influence of gravity tectonics has often been associated (Rutten, 1969).

The case shown in Fig. 4 – 18 illustrates the possibility of estimating the angle of shear resistance of a large slope with a typical slow creep movement. The figure shows a slope on a neovolcanic mountain range of Central Slovakia covering Tertiary

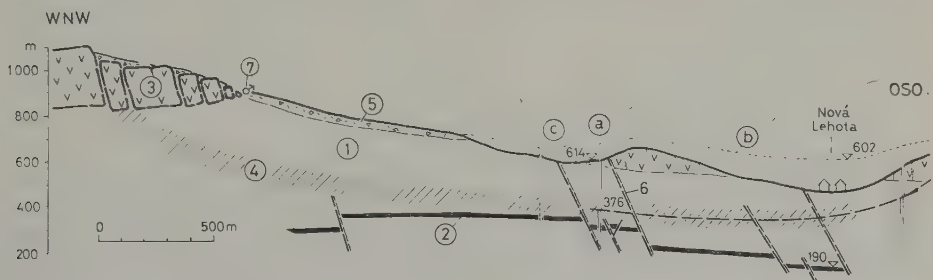


Fig. 4–18. Schematic cross-section of the Handlová coal basin in central Slovakia; 1 — overlying claystone, 2 — coal seam, 3 — andesite, 4 — assumed shear zone, 5 — slope debris, 6 — fault, 7 — spring, a — shaft disturbed over a depth interval of 376 to 447 m. (a) — profile across shaft a, dashed line (b) — profile 600 m north of (a).

deposits which contain coal seams. A slow creep movement caused by the height differences in the profile has been manifested by the fractures and failures in the range which have been known far back in the past. Two recent failures of shafts soon after they were constructed (one of them is shown in the unbroken line profile) indicated that there was ground movement. Only the shaft constructed in the line of the dotted line profile (a dividing line of the area) has remained intact. The general dip of the slope (from the top of the rock range to the potential outcrop of the shear zone) is of the order of 10 degrees in the unbroken line profile, compared with 7 degrees in the stable dotted line profile.

The ductile behaviour, however, cannot continue indefinitely. As mentioned in section 4.3, a prerequisite for this type of behaviour is a decrease in volume as the shear strain is taken up under the heavy overlying weight. This volume decrease must eventually cease, and then either the creep movement begins to slow down, or if the active forces increase for some reason (e. g. because of surface erosion), a thin (brittle) slip surface develops. In this case a thin active surface can be found within the disrupted and kneaded rock of the fossil shear zone.

4.7 Anisotropy of rocks

Another important factor governing the mechanical development of the slope movement is rock anisotropy.

As generally known, the shear resistance of many clays along the bedding planes is smaller than the resistance across the bedding planes. The difference is frequently of the order of 15 per cent. The orientation of bedding in Neogene clays is usually almost horizontal and the slip surface runs as far as possible parallel to the bedding (Fig. 4—19). The term „compound slide" has been used by Skempton (1969) for this type of slope movement. The safety factor isolines and the settlement isolines for a slope similar to that depicted in Fig. 4—1 (c) are shown in Fig. 4—19. According

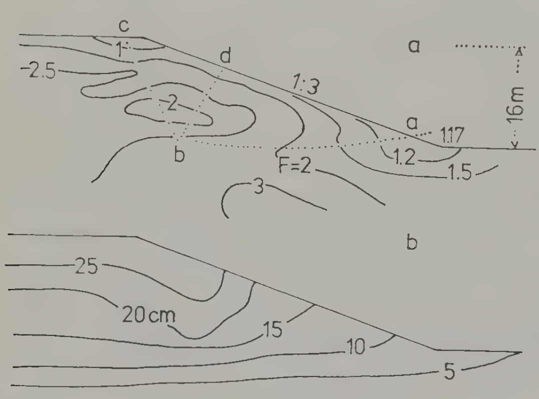


Fig. 4–19. Lines of equal magnitudes (a) of local safety factors and (b) of settlement for the slope shown in Fig. 4—1c, assuming a 25% decrease in shear deformation resistance along the horizontal bedding planes. This anisotropy has been simulated by inserting three horizontal layers of "weaker" elements.

to this analysis together with field obervations, the following points may be noted. The differences in the magnitudes of the safety factors along the potential slip surface are smaller than those in Fig. 4—1 (c), and therefore the tendency to progressive failure diminishes. This is probably the reason that the first symptoms of a possible failure often appear as fissures just behind the crown of the slope. The factor of safety of the slope analyzed in Fig. 4—19 is 1.7, but the slope probably will fail owing to subsidence of the wedge *cbd* and the possible penetration of rain-water into the slope. Deformation of the slope will also be triggered by frost action and by a weakening of the material at the toe where the local safety factor is only 1.17.

4.8 Arching in rocks

This is another factor that is involved in the development of slope failures. Rocks and soils are materials rigid enough to transmit shear forces, provided that the material is dense and strong enough to resist a decrease in volume under shear straining.

Rock arching around underground openings is widely known. The phenomenon can also be observed on slopes, where the curvature of the surface of weakness is not constant. This becomes increasingly important when abrupt changes in curvature or even breaks are present.

Fig. 4–20 shows the sensitivity of the magnitude of the vertical stress to breaks in the direction of a zone of weakness in a high rock slope. As an example of the arching

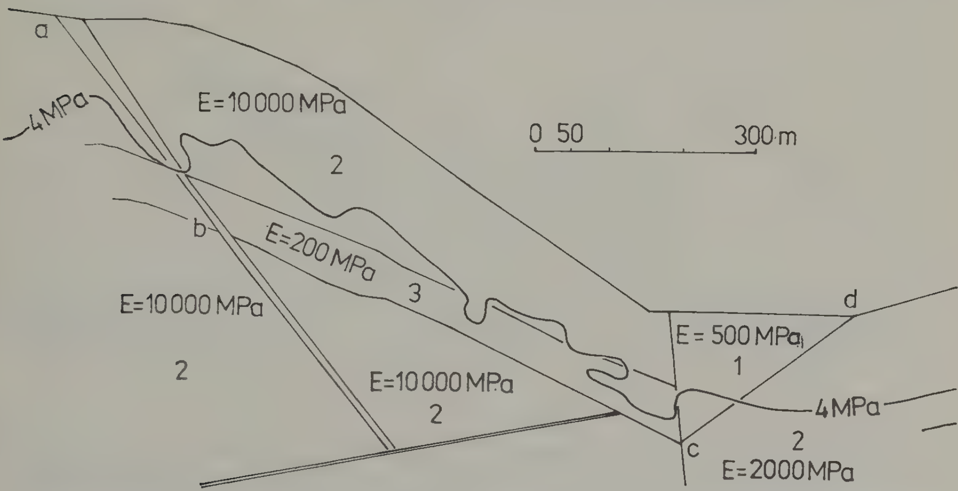


Fig. 4–20. Isoline of a vertical compression stress of 4 MPa in a slope formed of dolomite (2) containing a tectonized zone of older rocks under the valley floor (1) and a weak zone (3). The assumed deformation moduli are given. The development of the line of vertical stress in response to the zone of weakness *abcd* is observable (Mejzlík 1975, MS). The computed safety factor along *abcd* is 1.5.

of rocks above an abruptly curved section of the bedding planes, the collapse of a flank of Monte Toc into the reservoir of Vaiont can be cited. A working hypothesis of the mechanism of this landslide is presented in Fig. 4–21. As long as the sliding mass did not break by the development of tensile cracks (t) at the base and by the squeezing upwards of the wedges (k) in the upper compressed part of the mass, the rate of redistribution of the forces acting on the slip surface intensified. Owing to the stiffness of the sliding mass the normal compression forces became concentrated on the upper and lower areas of the slip surface. Fig. 4–21 shows the assumed distribution of the normal stresses along the slip surface.

Arching has, in principle, a beneficial effect on stability, because the arched body cannot move freely. However this effect is reduced by two phenomena, the first of which is the penetration of surface water into the sliding rock mass along cracks, resulting in an increase in uplift at the base. Secondly there is a decrease in the shear strength angle, which accompanies the concentration of normal forces on small areas of the slip surface. Therefore creep movement is to be expected. This is probably what occurred in the case of the Vaiont slide. Back calculation using Pettersson's methods yields $\varphi = 20$ degrees. After the redistribution had occurred, a value of $\varphi = 18$ degrees was sufficient for $F = 1$. Consequently, the slope movement changed in behaviour into a creep movement which has probably stopped and restarted several times. As soon as the rock mass began to disintegrate by the development of cracks

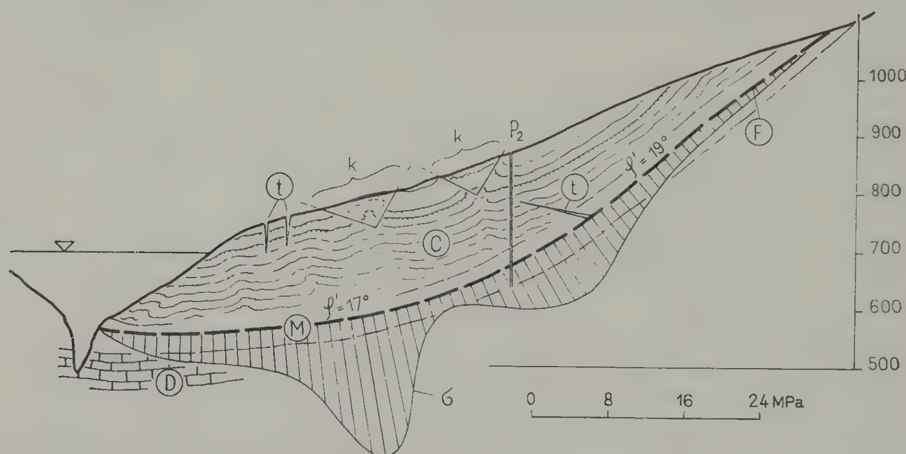


Fig. 4-21. Section through the Monte Toc slope as it was before collapse; **D** — flaggy limestone, Dogger, **M** — marly limestone, Malm, **F** — slip surface, **C** — marly limestone, Lower Cretaceous, **k** — rock wedges squeezed up, **t** — tension cracks (after Mencl, 1966).

and wedges, acceleration of the movement was resumed. The weight of an abruptly sinking rock arch might generate an increase in the uplift force of water accumulated at the base, thus adding to any decrease in stability. When examining the outcrops on the surface of the slip body (Selli and Trevisan, 1964) there are indications that the above processes have occurred at least once in the past.

4.9 Slope movements in weak rocks

The volume decrease associated with the shear straining of weak rocks is accompanied by ductile yielding (section 4.3), a phenomenon known as "plastic" behaviour in the common language of geology. The creep of debris or the motion of partially saturated mudflows serve as examples. Seasonal slope movements also show similar developments. The measurements referred to by Swanson and Swanston (1977) concerning the earthflow in the Tertiary pyroclastic rocks of the western Cascade Range at the Lookout Creek, Oregon, illustrate this mode of slope movement (Fig. 4–22). The annual rainfall is 100 to 200 cm, the time from December to February being the period of highest precipitation. The general slope angle is 13.5° (the earthflow is 900 m long), however the inclinometer tube was only installed near the toe. Unfortunately, no data on the water content of the material are indicated; such data would have been particularly interesting, because another large mass-movement of the terrain at Coyote Creek has displayed a distinct slip surface in the inclinometer tube.

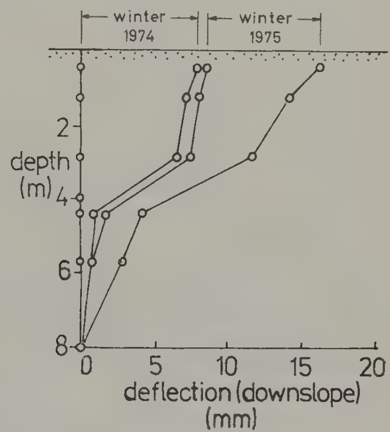


Fig. 4–22. Apparent seasonal deformation recorded in an inclinometer tube installed in the Lookout Creek earthflow, Oregon (Swanson and Swantson 1977).

4.10 "Multi-storied" sliding

As shown in Subsection 4.3, the ductile slope movements develop earlier as compared with the brittle sliding. Therefore, the surface movements of loose talus materials or of a weathered surface zone may occur first and the deeper sliding of a dense and less permeable rock may follow. In this way the phenomenon of a "multi-storied" sliding can develop. An interesting example was presented by Lee and Mystkowski (1979) from the brecciated and gouged broad fault zone of crystalline rocks in Colorado. The slope was inclined 28° and at least two slip surfaces or zones were found by inclinometer measurements at the depths of about 10 to 55 m. They were activated by the excavation for a highway. Several examples of a "multi-storied" sliding have also been noted by Ter-Stepanian (1974).

A "two-storied" sliding is shown in Fig. 4 – 12, but in this case it was caused by the loosening of the near-surface rocks resulting from the stress relief.

Chapter 5

GEOLOGICAL DEFINITION OF THE MAIN LANDSLIDE TYPES

5.1 Slope movements of surface deposits

This group comprises slope movements developed in the surface layers and produced by the activity of subaerial agents, by the character of slope deposits and by the topography of the slopes.

5.1.1 Talus creep and the terminal bending of beds

Talus creep is a slow downslope movement of rock debris, caused by subaerial processes. The loosening of rock fragments in winter accompanies the upheaval of the surface layers by freezing. In the spring thaw, the particles do not fall back to their original position but, under the influence of gravity, move slightly downhill. The movement of loose stone debris is caused mainly by temperature changes (expansion during heating and shrinkage on cooling).

Surface layers of clayey material move downhill as a result of slow plastic deformation (*creep*). Such a movement does not develop a discrete slide surface but rather a broader zone within which minute movements occur. The motion is restricted to a surface layer, the thickness of which does not exceed the depth of seasonal variations in temperature and humidity. The shifts are of the order of a few centimetres per annum but they may cause diverting of telephone and electric poles, and damage to retaining walls, buildings and underground lines.

The creep of slope debris was studied by Haefeli (1944, 1953), who likened it to the plastic deformations that occur in a snow bed on a mountain slope. In both cases, a slow downward movement is combined with a vertical movement of particles (subsidence). The rate of movement is greatest at the surface, decreasing towards the substratum. The density of the snow also increases with depth so that it is converted to firn and eventually to ice, unless the limit of stability is exceeded in which case the snow bed slips downhill in the form of an avalanche. Slope deposits likewise undergo compaction, although this occurs at a much lower rate.

The creep of detritus layers brings about a *terminal bending* of strata. Frictional force acting between the creeping detritus and the surface of the bedrock produces a gradual bending of beds. The dragged-out and disrupted ends of the bedrock layers become incorporated into the slope deposits, thus increasing their thickness.

Dragging out and bending of beds cannot occur on moderate slopes under present-day climatic conditions. These features date mostly from the Pleistocene, when even the hard rocks were so disturbed and shattered by recurrent deep freeze-and-thaw that they could be moved downhill.

The terminal bending of strata is widespread on slopes made up of shales, thin-bedded sandstones or limestones, weathered gneiss, granite and other rocks. Figure 5–1 shows a typical case of the bending of beds at a site on the outskirts of Prague, where this phenomenon is common in weathered Ordovician shales.

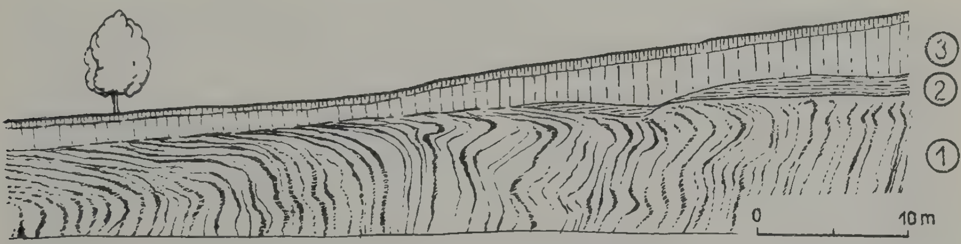


Fig. 5–1. Surface bending of beds in a loam-pit in Prague — Střešovice; 1 — weathered Ordovician shales, 2 — Cenomanian claystones, 3 — loess loam.

In excavation works the terminal bending of beds may be hazardous, because there is a high probability of sliding on the surfaces of dragged-out strata. In measuring the dip of beds in shallow test pits, care must be taken to avoid being misled by the false dip of any near-surface beds affected by terminal bending.

5.1.2 Sheet slides

The term sheet slides is used for the movements of shallow slope debris, loam and weathering materials on the surface of the bedrock. Slides of this type may be of large areal extent but their thickness is generally small, not more than a few metres. The disturbed slopes show several stages of sliding, ranging from initial fissuring of the surface layer up to advanced forms with several generations piled one on top of another. Sheet slides are generally at rest during dry weather, the movement resuming after heavy rainfall and during spring thaw, in particular after a long frosty period. The water rising by capillarity to the surface from lower unfrozen beds freezes to form a succession of thin ice laminae, which on thawing cause slaking of the surface layer.

Where clayey or marly rocks crop out on the slope surface, sheet slides develop by the slipping of weathered pelitic rocks along the unweathered bedrock. The weathered beds are often disrupted into small blocks separated by deep desiccation cracks formed by the shrinkage of clay in dry periods. As a result of alternating drying and swelling, the blocks become separated from the unweathered substratum. The rain-water fills the fissures, soaks into the rocks and causes swelling of the blocks along the fissures; this produces considerable horizontal forces that give rise at first to a characteristic undulation of the slope surface and eventually to slide movements.

Sheet slides of this type are common in those parts of the Cretaceous basin of Bohemia where clayey or marly rocks are exposed, and in the Neogene sediments of north-western Bohemia and north-eastern Moravia. They are a common feature in the Carpathian flysch regions (Fig. 5–2).

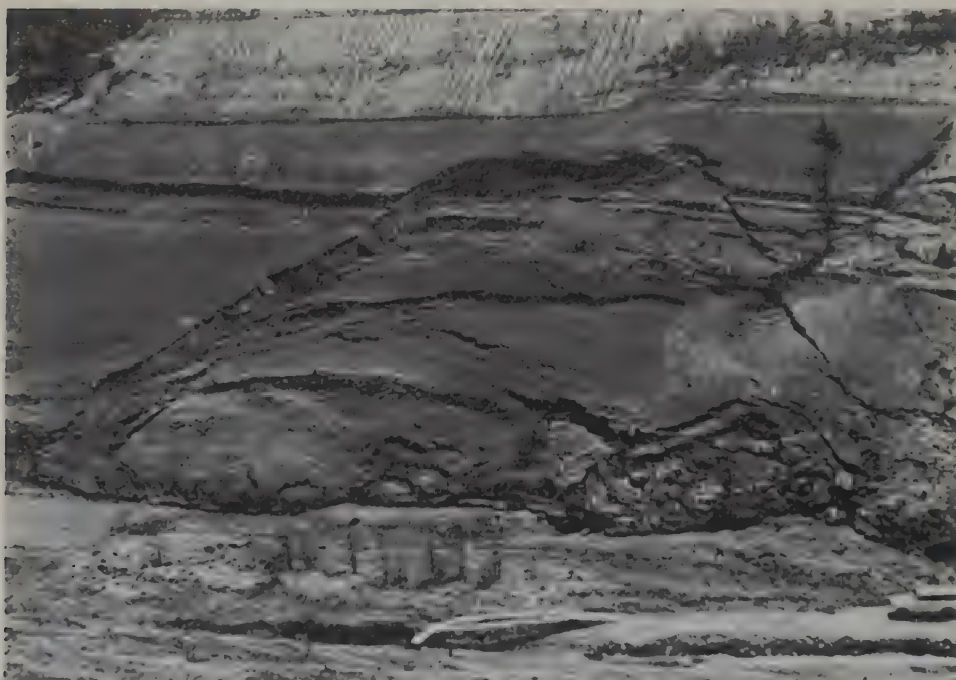


Fig. 5-2. A sheet slide in slope debris near Lazy, Carpathian Flysch Belt, Slovakia (photograph by Nemčok).

This type of sliding is exemplified by the landslide on the Chlomek ridge at Ctiměřice near Mladá Boleslav. The landslide occurred in 1926, a year of very high rainfall; the consistency of weathered sandy marls was disturbed by the water of several springs issuing from overlying weathered sandstones (Fig. 5–3). The excavation carried out in the loess loam at the toe of the slope was an additional factor leading to landslide. Waterlogged marls slipped down and destroyed two buildings. The slope was stabilized by means of several drains which removed the spring water.

An analogous sheet slide took place at Přerov nad Labem in the same year. The lower part of the slope of the Bílá hora hillock is composed of soft marl, whereas its

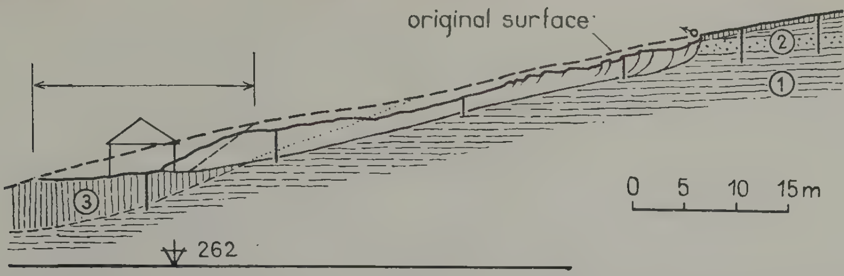


Fig. 5–3. Section through a sheet slide at Ctiměřice near Mladá Boleslav; 1 — Turonian marls, 2 — Turonian sandstone, 3 — loess loam.

top part is made up of solid flaggy marlstone of the Lower Turonian age (Fig. 5–4). At the foot of the slope there is a layer of marly slope debris 2 to 4 m thick. The sandy marlstones are jointed and permeable and a marked horizon of ground-water accumulates on the surface of the underlying impermeable marls.The beds dip gently (4° to

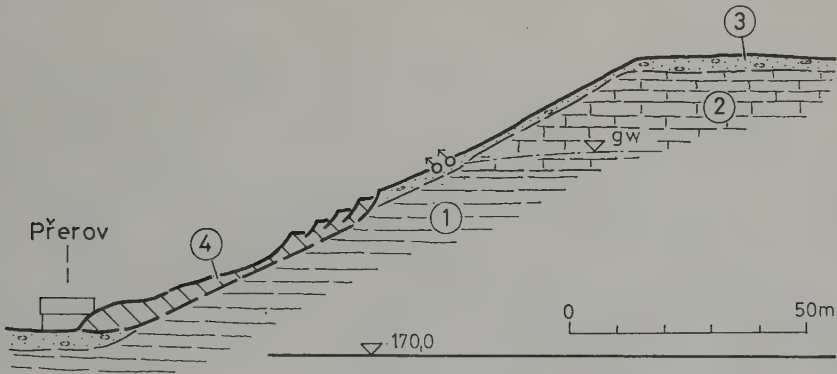


Fig. 5–4. Geological profile of the northern slope of Bílá Hora near Přerov nad Labem; 1 — Turonian marls, 2 — flaggy sandy marls, 3 — Pleistocene terrace, 4 — slipped material (Záruba 1926).

6°) towards NNE, and a number of springs arise along their contact on the northern slope. The most intense slide movement which occurred in 1926 demolished nine buildings which were situated near the foot of the slope.

5.1.3 Earthflows

This type of landslide usually occurs in the same areas as the sheet slides but under particular topographical conditions. The course and form of an earthflow is governed by the surface relief. Earthflows generally head in a large basin in the upper part of the slope where slope debris and weathering material is accumulated. Heavy rainfall may trigger the movement of this loose mass, which as a narrow flow follows an erosion gully or the channel of a brook, to form a loaf-shaped bulge at the foot of the slope. According to the material involved and its consistency, debris, earth or mud flows can be differentiated. Rain-water greatly increases the weight of the detritus and reduces its shear strength. There are also transitional types; rocks may tear away from the head scarp and slide down curved surfaces to the lower part of the slope, where the water-soaked mass moves as a flow.

Earthflows are abundant in the Flysch Belt of the Carpathians, from which a number of slope movements of this type have been described on both the Slovakian and Polish sides of the mountain range.

A typical earthflow occurred in 1937 near Dubková in western Slovakia (Fig. 5 – 5). The area is made up of the Tertiary flysch, consisting predominantly of marly and clayey shales with intercalations of glauconitic sandstone. The shales weather readily, so that the slopes are covered by a thick layer of clayey-sandy detritus which is susceptible to sliding. The earthflow originated in the upper part of a hill slope, in a depression drained by a small brook. This depression was filled with clayey-sandy

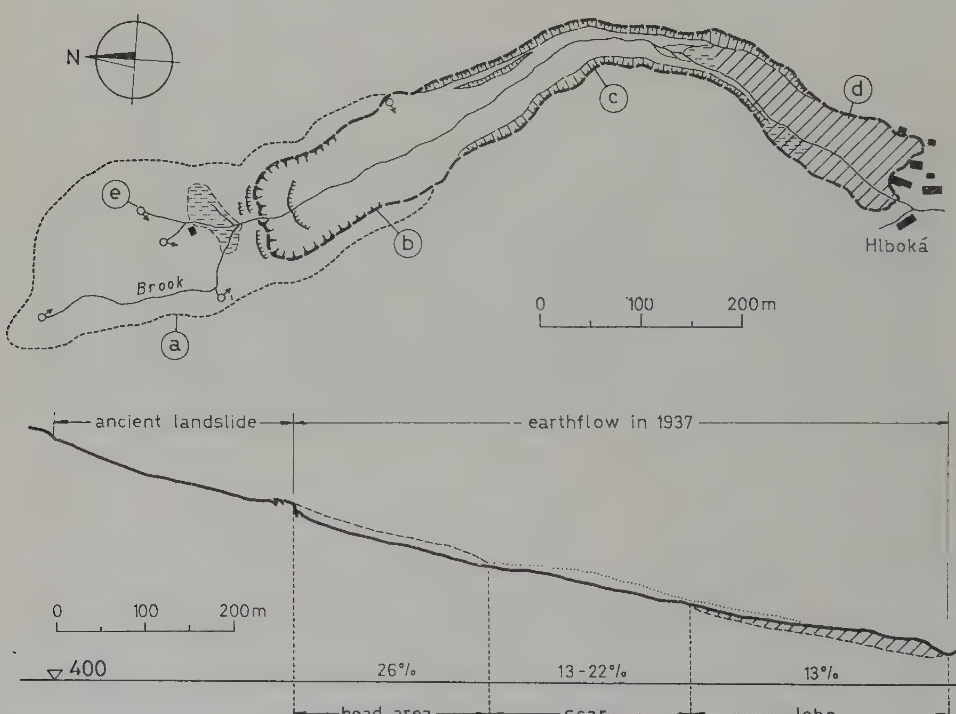


Fig. 5–5. Situation and profile of the earthflow near Dubková (Slovakia); a — head area of the ancient slide, b — earthflow of 1937, c — peripheral ridges, d — area of accumulation, e — — springs.

material up to 18 m thick, and its hummocky surface indicated that it was an old sliding area. In March 1937 a warm spell caused sudden thawing and this was accompanied by heavy rain. In the upper part of the slope a deep slump occurred and the loosened waterlogged material flowed down the channel of the brook (Figs. 5–6, 5–7). Within 24 hours about 120,000 m^3 material moved towards the foot of the slope, where it destroyed several farm houses of the village of Hlboká. The surface forms of the earthflow were very prominent, especially in the first days after the event. There was a steep head scarp 18 m high and the brook descending this slope in the form of a waterfall flowed into forested slope deposits which were torn into irregular blocks. The original erosion channel of the stream was widened into a broad U-shapped valley as is more typical of glacial valleys. On the smoothed valley walls there were striae in the direction of movement, and on either side of the earthflow lateral ridges were formed of clayey material and uprooted trees (Fig. 5–8).

Earthflows on mountain slopes, the head areas of which are filled with loose detritus, are a serious hazard to communications since remedial measures are difficult and expensive.

Fig. 5–6. View of the brook valley expanded by the earthflow near Dubková (photograph by Záruba).

Fig. 5—7. Earthflow near Dubková — slickensided walls of the erosion valley (photograph by Záruba).

The railway line from Púchov to Horní Lideč was constructed in an area where the rocks are susceptible to sliding. The earthflow near Dohňany (shown in Fig. 5—9) fills an old erosion gully developed between Palaeogene argillaceous shales and Lower Cretaceous (Neocomian) marly limestones. The contact between these two complexes is tectonic and the position of the erosion gully is governed by the low resistance of tectonically disturbed rocks to erosion. During a period of heavy rainfall the waste material became saturated, lost its stability and flowed like a stream down into the valley. The mass accumulated on the gravel alluvium of the floodplain of the Biela voda stream. At the time of construction of the railway the ancient landslide was not yet adequately stabilized, because its equilibrium state was disturbed from time to time due to the undermining of the toe by the river when its water level was high.

The railway-line was originally intended to run through a cutting which would have encroached upon the slide tongue and thus very probably would have disturbed the stability of the earth flow. The railway was therefore built on an embankment, erected on the sandy gravel of the flood-plain (Fig. 5—10). Water flowing from the head area and coming from springs and wet grounds on the northern margin of the slide, was discharged by drainage trenches. Undercutting of the slide tongue was stopped by diverting the course of the stream; the railway embankment also contributed to the stabilization of the flow. No movements have been observed since 1937 in this place.

Fig. 5-8. Peripheral ridges rimming the gully of the earthflow near Dubková (photograph by Záruba).

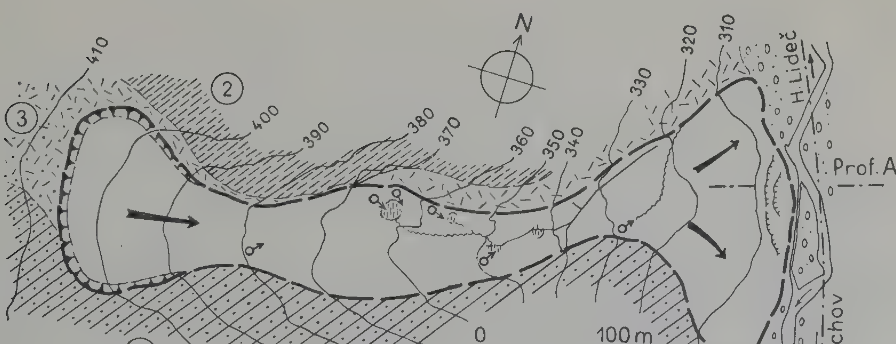


Fig. 5-9. Earthflow at Dohňany near Púchov (Slovakia); 1 — marly limestones (Neocomian), 2 — argillaceous shales (Palaeogene), 3 — slope debris, 4 — sandy gravel.

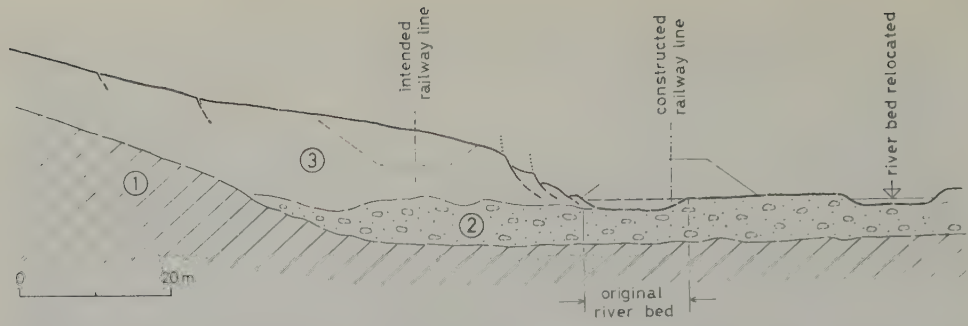


Fig. 5–10. Section A of the area of accumulation of the landslide near Dohňany; 1 — Palaeogene shales, 2 — sandy gravel, 3 — slid clayey debris.

Fig. 5 – 11 shows an earthflow in the valley of the Žarnovice stream. It was studied in detail since it had to be traversed by the Banská Bystrica – Diviaky railway line. The earthflow developed in an erosion furrow running between Triassic dolomites and andesite agglomerates, the head area lying about 100 m above the valley. The test-pits entered argillaceous shales of the Keuper below a thick layer of slope debris. Clayey waste and weathered tuff agglomerates moved in a narrow earthflow and formed a wide loaf-shaped tongue at the foot of the slope. The presence of argillaceous shales was obviously an important factor leading to a fairly large landslide in an area where slope movements are relatively rare.

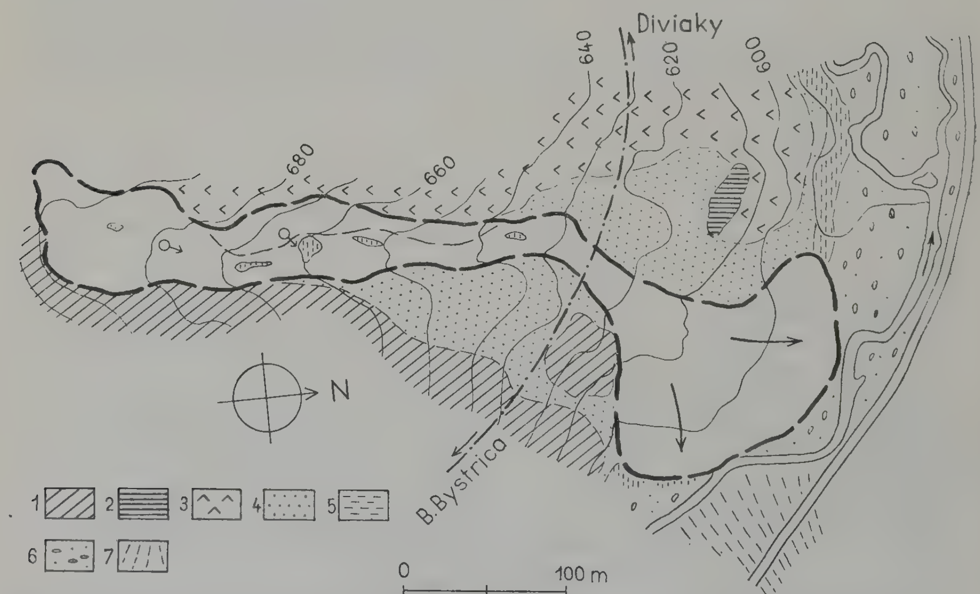


Fig. 5–11. Earthflow in the Žarnovice valley on the Banská Bystrica — Diviaky railway line. 1 — Triassic dolomite, 2 — andesite, 3 — andesite tuffs, 4 — slope loam and debris, 5 — marsh, 6 — sandy gravel, 7 — alluvial cone.

The accumulated mass of transported material came to rest on the valley floor on sandy gravels of the Žarnovice stream. Because of this, the mass was always well drained so that the tongue did not spread outwards. Subsequent movement only changed the relief of the flow, as additional material was heaped upon the older material in the form of bulges and ridges. The railway line traverses the landslide on an embankment and the slide area had been stabilized by a system of drainage trenches which discharged surface and ground-water.

5.1.4 The earthflow near Handlová

One of the largest landslides in Czechoslovakia occurred near Handlová in Slovakia in 1960. Using this landslide as an example, the development of earthflows, research methods and remedial measures will be discussed in detail.

In 1960 the slopes near Handlová were disturbed by extensive movements which destroyed part of the town and caused considerable economic losses. 150 houses were demolished, the main water supply and electrical conduits were destroyed and a main highway was disrupted. The main slope movements showed the characteristics of the earthflow and produced large deformations in the ground at the foot of the slope.

Fig. 5–12. Overall view of the Handlová earthflow from a helicopter (courtesy Geological Survey, Prague).

Earthflow I (Fig. 5–12) formed in December 1960 on the eastern side of the valley of the Handlovka river in a shallow gully filled with slope debris. From the topography of the slope above the present head area and from air photographs taken in 1955, it was apparent that it had been the site of sliding in the past. The movement involved debris of volcanic rocks and clayey and silty sediments of Sarmatian age which had already been drawn downwards by previous movements.

Subsequent movement occurred in the upper part of the course, where a wide head area developed. The rock debris and weathering material accumulated there saturated by surface water and emergent ground water, and flowed in the form of a narrow stream towards the valley. In the first phase of movement a wave of slushy material about 200 m broad and 18–25 m high extended almost to the highroad. The large load of the accumulated mass disturbed the equilibrium of a large bulk of slope debris piled up at the base of the slope, which resulted in a mass movement in front of the toe of the earthflow. This additional sliding accounts for an unusual widening of the flow at the base (up to 800 m or, combined with adjacent earthflow II, up to 1,200 m at the toe). The channel of the Handlovka river was blocked by downslipped material in several parts and the water level was raised by 5–8 m. The dammed water created several minor lakes.

Before the earthflow in 1960 the head area contained about 15 m of debris and weathering material. As a result of the movement, it was partly emptied so that in the spring of 1961 the thickness of the debris was only about 7 m. The length of earthflow I totalled 1,800 m and the volume of moving rocks was as much as 14.5 million m^3.

Earthflow II went into motion a fortnight later, at the beginning of January 1961, on the eastern side of earthflow I (Fig. 5–13). The head area of earthflow II developed in the contiguous gully which was filled with slope debris of similar composition to that of earthflow I. Earthflow II is about 1 km long and about 130 m broad in the upper part and 400 m at the toe. The volume of the sliding mass was approximately 5.7 million m^3, so that altogether more than 20 million m^3 of rocks were on the move.

Geological background. The basal part of the sliding slope is built up of Palaeogene pelitic shales and sandstone interbeds in typical flysch development. They are overlain by the Neogene complex with Tortonian coarse-grained tuffites at the base and the Handlová coal-seam formation. The coal seams thin out eastwards so that their thickness is reduced on the affected slope. Higher up, there are weakly consolidated clay-sandy sediments with intercalations of fine-grained sands. This Tortonian sequence is followed by the so-called Sarmatian Gravel Formation composed of sandy gravels

Fig. 5–13. Situation of earthflow I near Handlová in 1960–1961; 1 — vectors of points surveyed from January 1 to May 31, 1961, 2 — rubble work barriers in the Handlovka bed, 3 — destroyed buildings, 4 — hydrogeological borings from which water was pumped by means of submersible pumps; 5 — large springs in the head area.


brook 1
A
B
III
C
3
D
brook 2
landslide I
landslide II
II = E
7
7'
brook 3
Geodet. profile I
p5
R. Handlovka
Handlová
30
31
11
12
33
34
1
2
3
4
5
0
100
200
300 m
N

and clays. The top part of the slope is built up of thick sheets of andesite and andesite agglomerate.

The steep rock walls exposing the margins of volcanic sheets suffered subsidence which took place mainly in the Pleistocene as a result of the squeezing out of the plastic substratum. The margins of sheets were broken into large blocks which sank into weak pelitic rocks and moved towards the valley. Isolated andesite and agglomerate blocks torn off the originally continuous volcanic sheets and scattered over the slopes of the Handlovka valley stand as evidence of these ancient movements (Fig. 5–14). The sinking and slipping of marginal blocks occurred simultaneously with

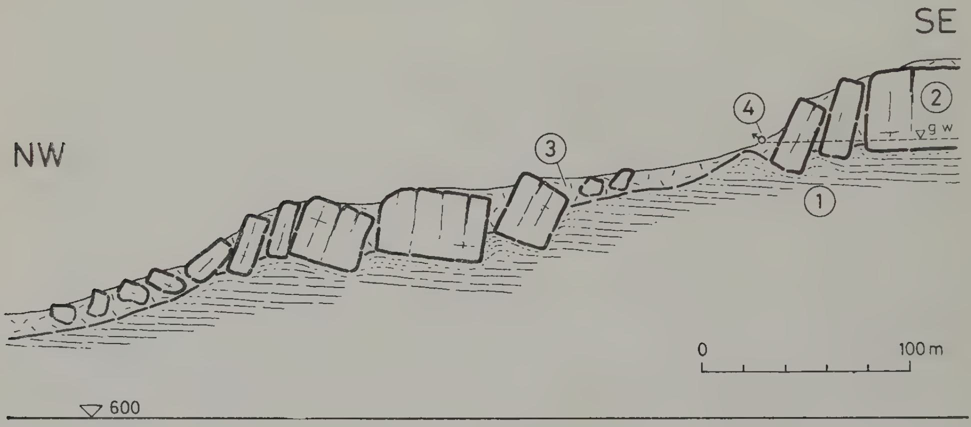


Fig. 5–14. Geological section of a block slide at the margin of the andesite sheet near Handlová; 1 — Neogene clays and silts, 2 — andesite, 3 — andesite debris, 4 — springs.

downcutting of the Handlovka river, which contributed to the unloading of soft Neogene sediments and, consequently, to the gradual squeezing out of these sediments from under heavy blocks of volcanic rocks. It cannot reliably be ascertained whether the blocks were sinking by their weight, or whether the periglacial climate effected a change in clay consistency.

Causes of slope movements. In the Handlovka basin the geological and topographical conditions are extremely favourable for the initiation of slope movements. In the upper part of the slopes, the mode of deposition of the Neogene rocks is not conducive to slope stability; plastic pelitic sediments are overlain by heavy volcanic rock divided into large blocks by vertical joints at the margins of the sheet.

Susceptibility to sliding is also increased by the heterogeneity of slope sediments, which in some places are permeable, composed of sand and rock fragments, and in others consist predominantly of clay beds. Both slope deposits and weathered pelitic shales have been disturbed and pulled out by ancient slope movements. New movements develop mostly on previous slide surfaces.

Topographical conditions are an important factor which largely controls the course and extent of landslides. The gradients of slopes formed of semi-consolidated clayey sediments would have been reduced long ago if their upper edges had not been protected from denudation by sheets of solid volcanic rocks. The juvenile relief of the Handlová area has also been influenced by young tectonic movements manifested by the subsidence in the lower reach of the Handlovka river, which consequently has a steep and ungraded course. The transportation power of the river is another contributive factor as this is sufficient to remove the slipped rock mass. Undercutting at the foot of the slope disturbs the stability of debris, especially in those parts where steep concave banks develop.

No less important are the hydrogeological conditions. The alternation of permeable and impermeable rocks throughout the slope is responsible for the existence of several overlying ground-water horizons, viz. in the jointed volcanic rocks above the impermeable clays, in the Sarmatian gravel and sand deposits and in the weathered surface layers of Paleogene shales. Discontinuous pervious layers of waterlogged debris also alternate with clayey soils in the slope deposits. Spring water issuing at the base of jointed volcanic rock, seeps together with rain-water into the slope debris, changing its consistency and impairing its physical properties. In addition, uplift occurs during periods of heavy rainfall in those permeable beds which have a less permeable roof, and this decreases friction at the slide surfaces.

The slide movement of 1960 was triggered by a large increase of the water inflow into ancient slide masses, following unusually high precipitation. According to the records of the ombrometric stations in Handlová and Prievidza, the precipitation of that year surpassed the mean value for 1901 – 1960 by nearly 50 per cent, the major portion of it falling in the autumn and winter months; the fifty years' average was 689 mm, while in 1960 the rainfall in the Handlová area amounted to 1,045 mm.

Moreover, the old drainage works which were relied upon for a more rapid discharge of surface waters were neglected, so that, for example, the brook numbered 3 (Fig. 5 – 13) which drained quite an extensive area to the east of earthflow II, discharged its water into the peripheral crack of this slide. One of the first corrective measures was concerned with diverting the stream water back to the original channel. The seepage of water had also increased in the immediately preceding years because the slopes which had been spoiled by fly-ash, could no longer serve as pasture land and ploughing had disturbed the grass cover.

Reconnaissance and improvement operations. Immediately after the slope failure, geological research and corrective measures were undertaken. At first, the progression of the landslide and deformations of threatened buildings were carefully observed so that rescue and evacuation might be organized in time.

The systematic measurement of the rate of movement of the landslide started on December 22, 1960, i. e. about 10 days after the first indication of sliding appeared in the head area. Nine geodetic profiles and a chain of points along the highway were established, movements were checked every fourth day and the chain was monitored every other day. In Fig. 5—16, several of these profiles are shown and the movement of individual points between January and the end of May 1961 is indicated. The largest horizontal shift (240 m) was measured in profile III, in the middle of the earthflow. In the area of deposition horizontal displacements were of a minor extent (106 m in profile II and 38 m in profile I). The greatest horizontal shifts measured in the chain amounted to 22 m (Bajtoš 1961).

In the first days the rate of the earthflow movement was as high as 6.30 m in 24 hours (in profile III). The rate gradually decreased (Fig. 5—15) during the long

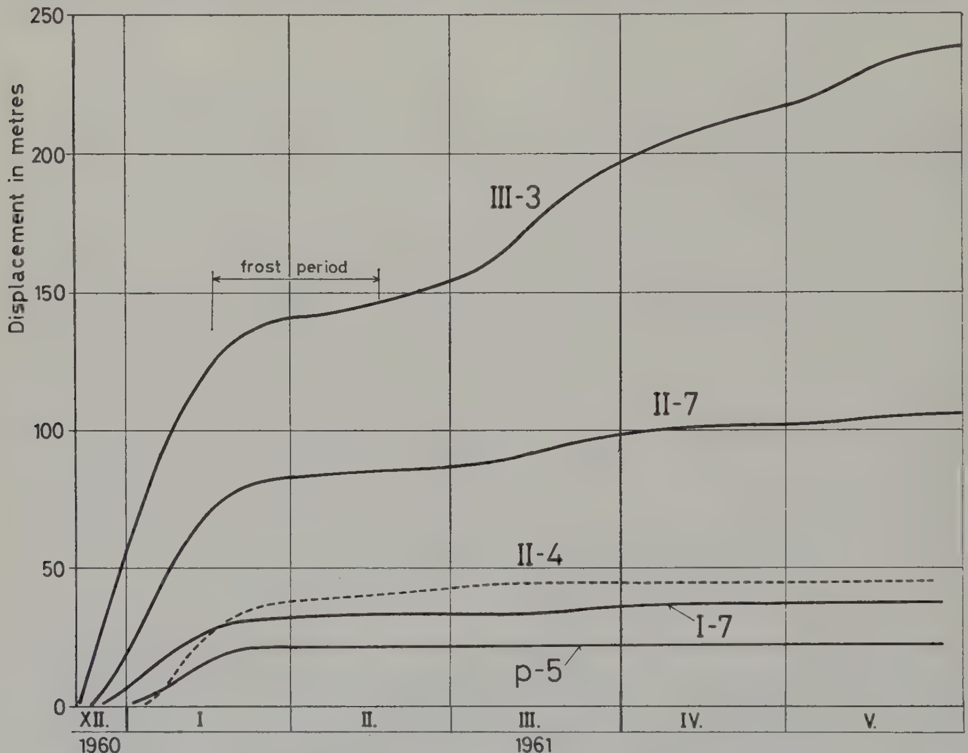


Fig. 5-15. The rate of movement of several points in profiles I, II, III, between December 1960 and May 31, 1961 (after Bajtoš, 1961).

period of freezing, and in the summer months of 1961 the movement ceased altogether. Although the rate of sliding was high during the first days, it was possible to organize the evacuation of threatened buildings so that there were no casualties. The houses at first showed severe cracking and finally collapsed after only two or three days.

Measurement of transverse profiles. The nature of the landslide is clearly seen from the profiles which were surveyed and geologically characterized in April 1961. The profiles were chosen so that the original ground surface and the probable course of the slide could be reconstructed. The analysis of the positions of the cracks and the individual blocks made it possible to differentiate in the profiles between the main earthflow and subsidiary lateral slides which developed gradually on either side of the head area, and in places where the ancient valley was deepened by the erosive effect of the earthflow.

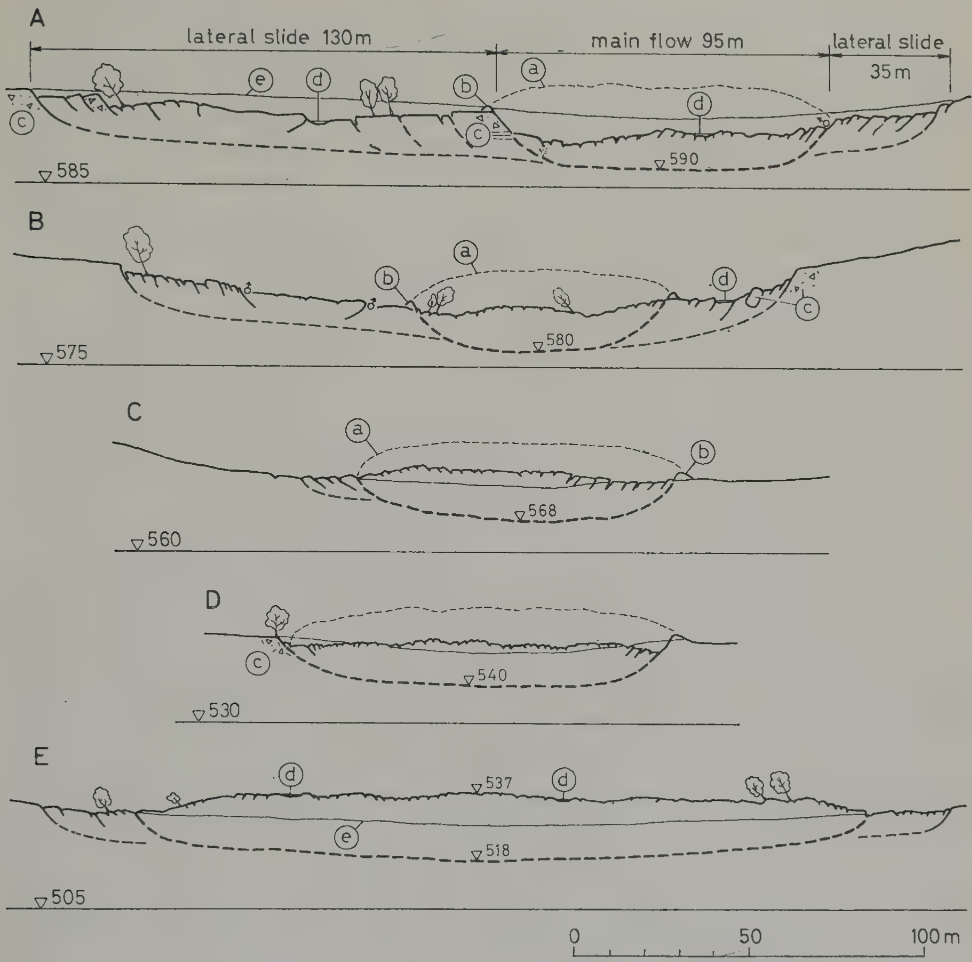


Fig. 5–16. Transverse sections of earthflow I near Handlová; a — the highest level of the earthflow, b — squeezed out ridges, c — andesite debris, d — lakes on the slide surface, e — original slope surface.

Profile A (Fig. 5—16) is drawn across the slide, about 380 m from the upper edge of the head area. The main earthflow 95 m wide at this position was bounded by distinct longitudinal cracks. The highest level of accumulation was 613 m a.s.l. towards the end of December 1960, as indicated by the lateral ridge heaped up on the left margin of the earthflow. By April 1961 the head area was considerably emptied and the surface level of the flow dropped 12 m. In the left side of the scar, loamy-sandy debris with andesite blocks above slickensided greenish-grey clays of Sarmatian age were found. Towards the end of March and at the beginning of April 1961 several lateral slides started which extended the head area by 130 m on the left side and 35 m on the right side; the width of the head area in profile A totals 260 m.

Profile B (Fig. 5—16) was surveyed across the earthflow 110 m below profile A. The main 75 m-broad earthflow is distinctly delimited by several longitudinal cracks filled with water-saturated clay. The shape of the lateral ridge preserved on the left margin indicates that the slipped mass reached an elevation of 604 m towards the end of December 1960. In April, the surface level of the earthflow was substantially lower because most of the material had already reached the foot of the slope. After the deepening of the gully, the slide spread progressively to either side, by 82 m on the left and by 35 m on the right. In the steep scarp of the head area a block of agglomeratic tuffs with andesite boulders was exposed on the right.

Profile C was surveyed 170 m below profile B, the profile line coinciding with geodetic profile III. The main 94 m-broad earthflow was distinctly delimited by a system of slip surfaces along which the slipped material was liquefied. In January 1961, the earthflow level was about 12 m above the adjacent surface. On its outer edges the clays were vertically striated owing to transverse basculation of the slide body. On the left side were raised blocks of fen overgrown with horsetail (Equisetum fluviatile). A lateral slide about 17 m broad developed to the left of the main earthflow. The surface of the main earthflow was moderately bulged so that brooklets discharging the head area flowed in channels on either side of the earthflow.

Profile D was drawn across the slide 240 m below profile C. The width of the flow at this point was 110 m and in April 1961 its margins were sunk below the original level of the terrain. Lateral ridges up to 3 m high, preserved on the right side indicate that the greatest height of the moving mass was approximately 23 m (above the slide plane) towards the end of 1960. At the surface, the flow was broken into irregular blocks which at the margin were aligned to form a longitudinal wave pattern. The

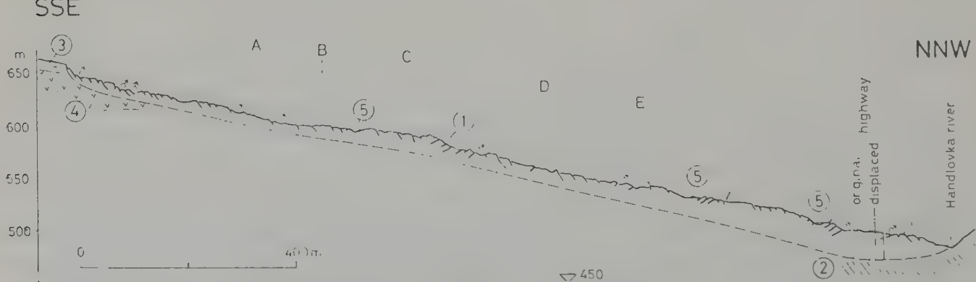


Fig. 5–17. Longitudinal section of earthflow I near Handlová; 1 — the highest level of the earthflow, 2 — marly shales (Palaeogene), 3 — slope debris, 4 — sandy gravel with interbeds of clay and tuffite, 5 — lakes on the slide surface, A — E — transverse sections in Fig. 5—16.

middle part of the flow is moderately bulged. Streams of water were confined to the marginal cracks.

Profile E was surveyed in the line of geodetic profile II in the upper part of the area of deposition, where the moving mass accumulated to a height of 8 – 10 m above the original ground surface. The width of the flow was 210 m; on either side there are lateral slides pulled down by the huge mass of the main flow.

Longitudinal profile I (Fig. 5 – 17) was drawn approximately through the centre of the head area and down the earthflow as far as the transverse profile E. At their intersection the longitudinal profile was bent so as to run perpendicular to the course of the Handlovka valley.

In the head area as far as profile B the gradient is 12.6%; between profiles B and E it decreases to 10.6%. The stretch between profile E and the highway slopes at a minimum gradient of 7.2%. Thus the downward moving masses accumulated in this area and from the front edge of the mass subsidiary slides moved down, blocking the Handlovka channel locally.

The borings V-116 and V-117 sunk into earthflow I about 30 m above and below the highway revealed that the river valley floor had originally been deeper and had been gradually raised by sliding material which pushed the course of the stream towards the left bank. In 1960 the level of the Handlovka was about 12 m higher than the deepest level indicated by the sandy gravel encountered in the borings. This would account for the ungraded slope of the stream in this stretch. The displacement of the channel to the left bank brought about an increased rate of lateral erosion, which in turn initiated a movement of the left valley slope.

Corrective measures. The first task of the corrective operation was to divert the water flowing into the slide area and to catch up and drain the springs issuing in the head area. This was accomplished by ditches, wooden troughs and auxiliary pipes. Simultaneously, water was pumped from all existing wells.

When earthflow II began to move towards the new part of the town and the railway line, the remedial measures were aimed mainly at stabilization of the flow. In the area of deposition, several large-diameter borings were drilled for the purpose of continuous pumping of water, and six galleries were driven into the frontal part of the slide. As the timbering was stressed by high vertical and longitudinal pressures, the driving of the galleries met with considerable difficulties. The galleries were 100 – 190 m long and terminated in the Paleogene shales, some of them branching at the end, with a backfilling of andesite material. Drainage pipes were laid along the bottoms of the galleries and vertical borings provided effective drainage of the overlying waterlogged debris. From those galleries discharging water from the toe of the slide valuable information was obtained on the composition of Quaternary sediments.

The sections through the galleries have revealed that the slope movements recurred several times since the Middle Pleistocene. The slipped masses are composed of agglomerate and andesite debris with moulded beds of green-grey silty clay of the Gravel Formation. These Neogene sediments were displaced from the upper part of the slope onto the Palaeogene shales and the flat cones of sandy gravels spread on the Handlovka river floor in the widened parts of the valley.

From evidence obtained from the remoulded greenish-grey silty clays, it seems that several flows were piled one on top of another. In gallery 12, four layers of

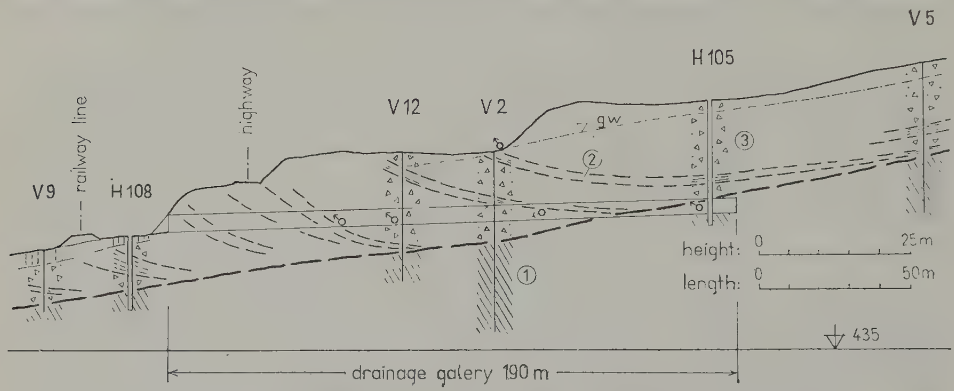


Fig. 5-18. Section through drainage gallery 12; 1 — argillaceous shales (Palaeogene), 2 — moulded clayey silts, 3 — loamy debris with andesite blocks.

Fig. 5-19. The head area of earthflow I near Handlová (photograph by Záruba).

andesite debris alternate with moulded clay beds (Fig. 5–18). As the debris is permeable and the silty clays are almost completely impermeable, several horizons of ground-water formed at the toe of the slope. The water was under artesian pressure in some places. As part of the corrective operation, these horizons were drained by boring wells so as to stabilize as quickly as possible earthflow II and prevent it from threatening the railway line and part of the town. An effective discharge of the ground-water horizons was achieved by drainage borings opening into drainage galleries.

The driving of the galleries made it possible to conduct hydrogeological observations. The ground-water horizon at the Palaeogene-Quaternary boundary proved to have a very high yield, the water having accumulated mainly in the weathered and jointed surface layers of Palaeogene shales. In gallery 12 a strong inflow of water appeared in the fissured Palaeogene rocks (initial yield: 100 l min^{-1}) at about 72 m from the entrance. In gallery 31 an inflow of water of up to 40l min^{-1} was measured as soon as weathered Palaeogene shales were encountered. After galleries 11 and 12 had been completed, the ground-water table in the neighbouring boreholes dropped to the level of the gallery floors. The galleries and systematic pumping discharged a large amount of water from the permeable beds of the sliding masses as well as from the superficial Palaeogene beds forming the substratum of the earthflow (Figs. 5–19, 5–20).

Fig. 5–20. Lateral ridges of the earthflow near Handlová (courtesy of the Geological Survey, Prague).

As the channel of the Handlovka river was constricted in several places by the slipped mass, several rubble work barriers were built in the threatened sections of the stream bed in order to stabilize the channel and reinforce the banks against erosion. Later on, the Handlovka waters were transferred through an underground pipeline and the valley was filled with spoil from mines.

In the summer months after partial stabilization of the earthflow had been achieved, paved ditches for the permanent drainage of the entire slide area were begun. Ditches constructed of concrete blocks bedded in gravel proved adequate during the first two years but they were eventually disrupted here and there by torrential spring rains and grazing cattle (Fig. 8–8). Generally in such cases the continuous maintenance of corrective measures and installations can not be neglected. The remedial treatment also included a systematic afforestation of the slide area with suitable species of tree.

5.1.5 Debris flows, Muren

The term debris flow refers to the rapid movement of water-saturated debris and soil by true flow process. Movements of this type arise on steep mountain slopes in loose, little consistent rocks, being induced by sudden floods. Mountain debris flows are referred to by a local Alpine name, *muren*, or as *debris avalanches*. They commonly originate above timberline in gorges filled with rock fragments. During torrential rain, the unsorted material ranging from fine sandy detritus to large boulders can be swept downhill, the ratio of solid particles to water usually being 1 : 1. Muren move very swiftly, passing trains have been known to be caught and buried by this type of flow (Fig. 5–21). In temperate zones muren tend to be confined to high-mountain areas, nevertheless they can be initiated by careless disturbance of conditions on any slope covered with loose detritus, for example by clearance or disturbance of the vegetation cover, or by leaving the slopes of deep road and railway cuttings ungrassed for a long time.

Some dejection cones consist of material from several successive debris flows, the younger overlapping the older ones. This is the case of the dejection cone below Mt. Slavkovský štít near Smokovec in the High Tatra Mountains. The "collecting" area of the rock debris extends over the steep southern slope of Slavkovský štít above the treeline; the granite debris was washed down during torrential rain and was deposited in the form of longitudinal ramparts on the flat dejection cone above Smokovec. The debris flows have been surveyed, both in terms of height and situation, and Fig. 5–22 shows that some of them partly fill the erosion gully of the stream while others have built ramparts several hundred metres long on either side of it. From the disposition of the debris flows it may be inferred that the intensity of torrential rains gradually decreased, since the oldest debris flows are the longest, the younger ones extend only to the upper parts of the debris cone. The composition of the old debris

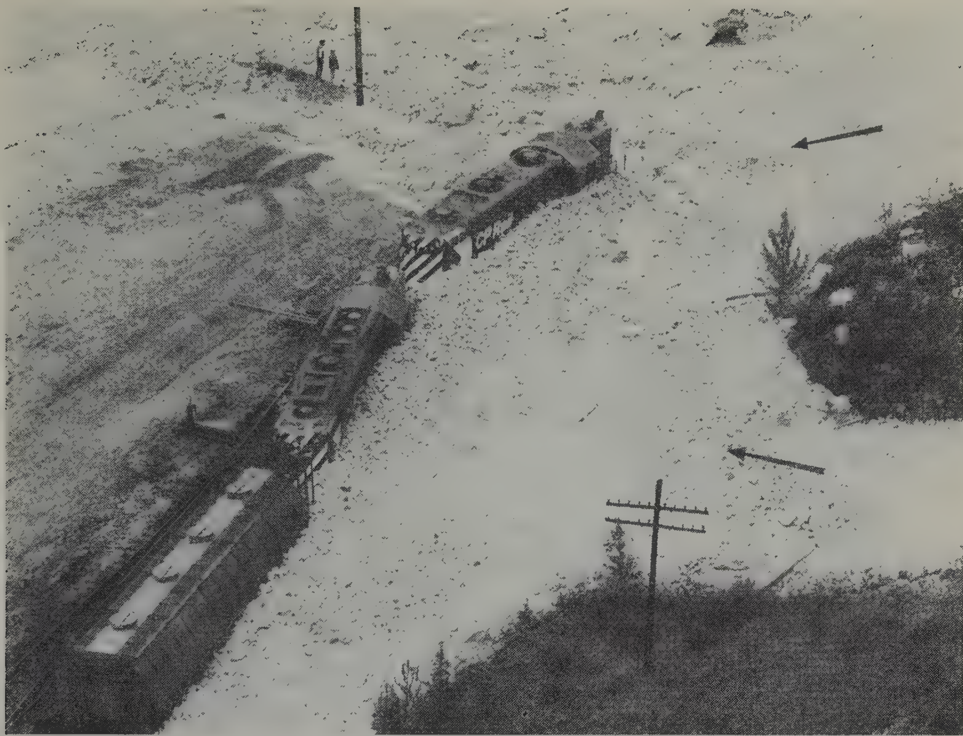


Fig. 5–21. Debris flows have repeatedly blocked the Canadian Pacific Railroad. The slide of Sept. 1978 caught a freight train, which dammed debris flow deposits in the Kicking Horse valley, British Columbia (courtesy of the Calgary Herald).

flow can be seen in Fig. 5 – 23 taken during excavation of the foundation for a sanatorium in Smokovec. The material is unsorted, comprising large angular or little-worn boulders and fine sandy detritus.

Debris flows are more destructive in arid and semi-arid regions, where the rock detritus, loam and mud are swept down valleys during intermittent torrential downpours. In these regions the ground surface is not protected sufficiently by vegetation against strong rainwash. The development of loam deposits on the flood plains in Central Europe may be interpreted by analogy with these circumstances. Thus, the loams may have been washed down and deposited after the rather sudden deterioration in climatic conditions about 800 B. C., at the beginning of the Subatlantic period. Disturbance of the vegetation cover had already occurred in the form of extensive clearance and cultivation activity; numerous settlements from the Bronze Age are buried by these flood loams.

Volcanic mudflows are of a similar nature. Volcanic explosions are generally accompanied by heavy rains, which wash the loose ash and minute ejecta as a mushy mass down to the foot of the volcano. Such a volcanic mudflow buried the town of

Herculaneum during the eruption of Vesuvius in A. D. 79. Debris flows also form on the slopes of dormant volcanoes covered by mountain glaciers, when volcanic activity is rejuvenated. The glaciers melt rapidly with the heat of the eruption, and the flood waters carry down the slope debris and morainic material.

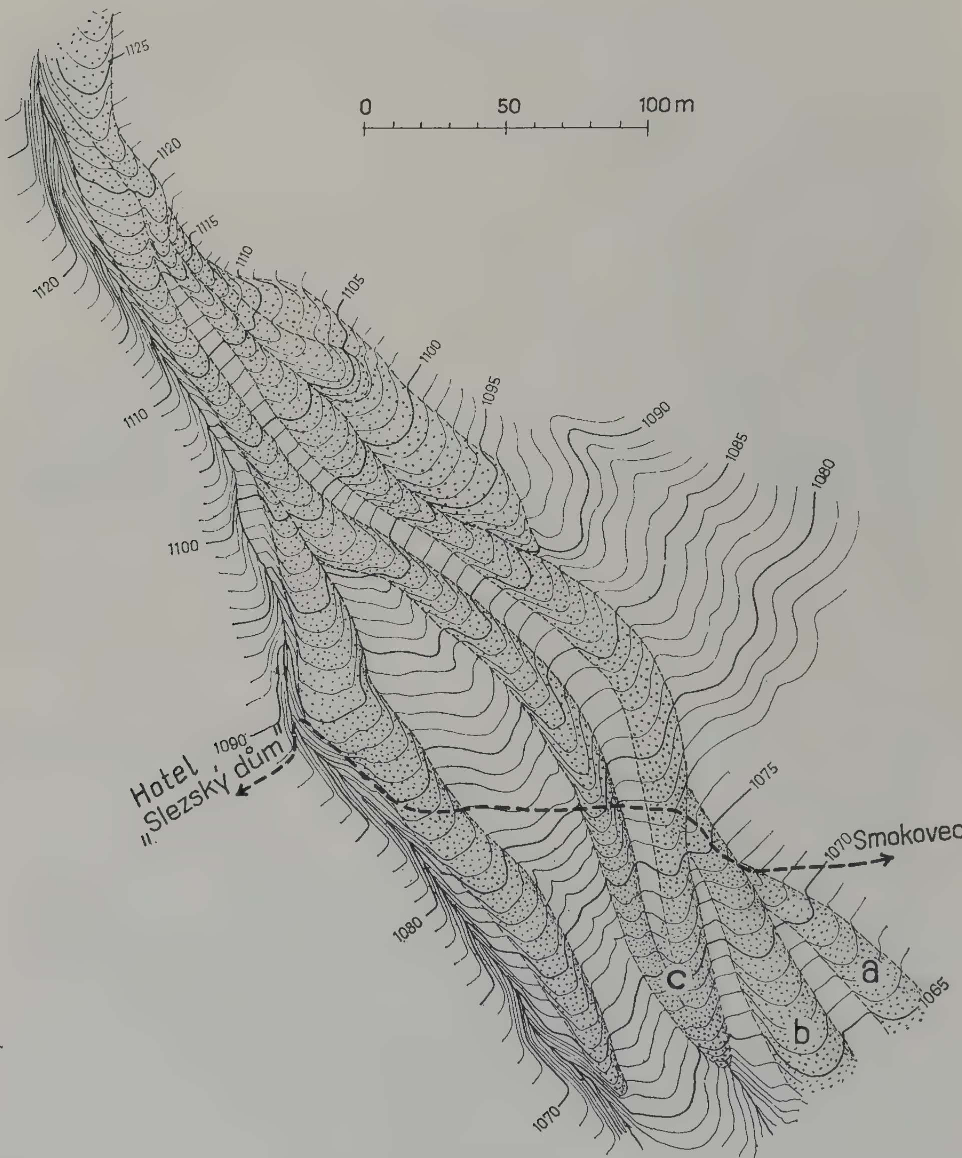


Fig. 5–22. Ancient debris flows are easily discernible at the surface of the dejection cone below Mt. Slavkovský šít (High Tatra Mts.); the oldest is debris flow (a) and (c) is the youngest.

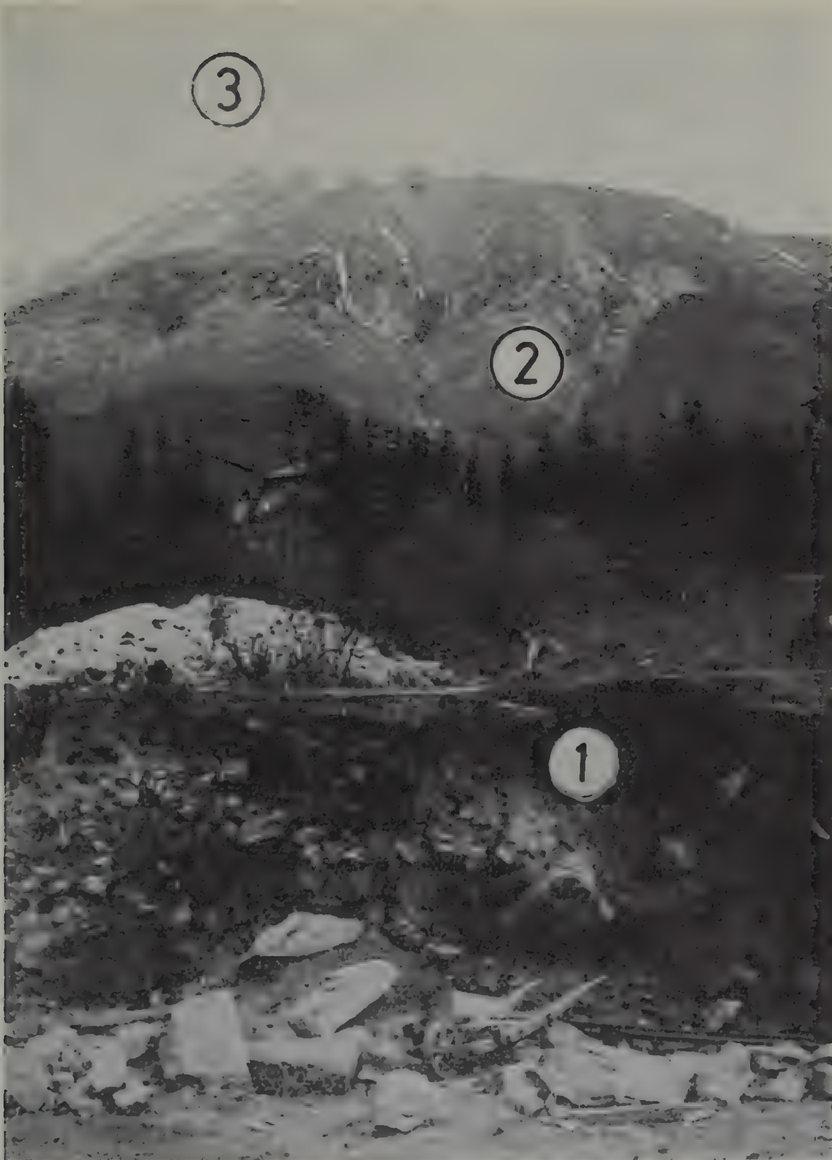


Fig. 5-23. A debris flow exposed in the foundation for a sanatorium in Smokovec below Mt. Slavkovský štít (3); 1 — ancient debris, 2 — catchment area above the treeline.

Slope disturbances produced by *liquefaction of sand* can be included within this group. The liquefaction of sand may occur as a result of the forces developed by the water streaming through the sand. Water percolating through a sandy soil must overcome the resistance to flow in the intergranular spaces. When this happens, the pressure head is diminished by Δh (Fig. 5–24) and the pressure difference is then held by the sand grains. If the pathway in the direction of flow is Δl (cm) and the loss of piezometric head is Δh (cm), then the flow of water exerts a pressure i (0.01 N) on the grains (per 1 cm^3 of soil volume) in the direction of flow; i is the hydraulic gradient in the direction of flow and equals $\Delta h / \Delta l$. From this it is evident that the pressure produced by water percolating through sand is proportionately greater when the difference in water levels over a short distance is greater. Therefore the tendency to

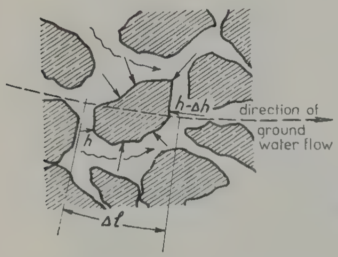


Fig. 5-24. The origin of running sand caused by percolating water.

liquefaction is greater in fine sand than in coarse sand and gravel, since in the latter situation the ground-water table generally has a smaller gradient. Liquefaction of sand may be provoked by an abrupt lowering of the water level in a reservoir, or by the puncturing an impermeable cover over a sandy aquifer.

In artificial cuttings, this type of slide seldom occurs because the ground-water table in permeable soils generally drops with the progressive deepening of the cutting. The liquefaction of sand can occur, however, when a water-saturated sand bed is laid bare by the removal of low-permeable rocks. The diagram in Fig. 5–25 shows

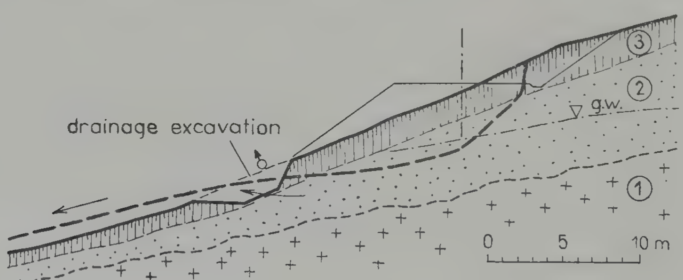


Fig. 5-25. Ground water stored in sand (2) under a layer of clayey slope loam (3) gushed out after the excavation of a trench; the water current washed out the sand and the slope was severely disturbed; 1 — granite.

a similar slope failure from the railway construction in Moravia. An old erosion furrow in the granite was filled with sand washed down from the higher part of the slope and had been finally covered by slope loam and humus. Because of the low permeability of the cover, water accumulated in the sand under artesian pressure. The excavation of a drainage trench under what was to be the embankment exposed the water-saturated sand that flowed out and produced an extensive slope failure. In such a case, the correct procedure today would be to drain the ground water by horizontal borings.

Loose, highly porous sands may also be liquefied by vibrations, which bring about some re-arrangement of the sand grains; in this way the density of the mass becomes greater and the porosity smaller, the superfluous water being expelled from the sand. Since the water cannot escape immediately, the pore-water pressure increases, reducing friction between the grains, and the soil becomes liquid for a brief period.

This type of slope failures occurs relatively frequently on seashores (e. g. in the Netherlands) or on lake shores. Analogous slides may occur on artificial slopes, such as poorly compacted sandy embankments, the banks of reservoirs, the upstream faces of earth dams, etc. As mentioned above, these slope accidents are caused by the high porosity of sand, and therefore fills built of sand must be very thoroughly compacted.

"Running" sands are very dangerous in coal basins, because they may cause the caving-in of mine workings and the sudden subsidence of the ground surface in an undermined area.

5.2 Landslides in clayey rocks

5.2.1 Landslides along rotational slide surfaces (slumps)

If the shear strength of unconsolidated or poorly consolidated clayey rocks is overcome, deep-reaching slides form along curved slide surfaces in the slope. Slides of this kind are called slumps. In homogeneous clayey rocks (clays, claystones, argillaceous shales) the shape of the slide surface is more or less cylindrical. From theoretical computations and measurements a somewhat different shape of slide surface might be assessed, but the difference this makes to the stability calculations is small – within the range of reliability of the method used in the determination of soil strength.

The slumps have a characteristic form (Fig. 5 – 26). Since the slide surface is curved, the movement involves rotation and the surface of the slumped mass is usually inclined into the slope. The head area*) shows a typical concave form and the moving

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* The terminology used to describe the individual parts of rotational slides is not uniform. In the literature different terms can be found for the upper part of a slide: e. g. the head area (Varnes 1978), root area (Záruba and Mencl 1969), source area or scarp area (Cruden 1980). In this book the term "head area" will be used which we think to be preferable to the other terms. Two useful descriptive terms have been introduced by Varnes (1978): zone of depletion for the area within which the displaced material lies below the original ground surface, and zone of accumulation for the area within which displaced material lies above the original ground surface.

material accumulates at the foot of the slope. Transverse cracks which open in the tongue, gradually become filled with water so that the equilibrium of the slope is further disturbed; the slump mass is often so waterlogged that the tongue takes on the character of an earthflow.

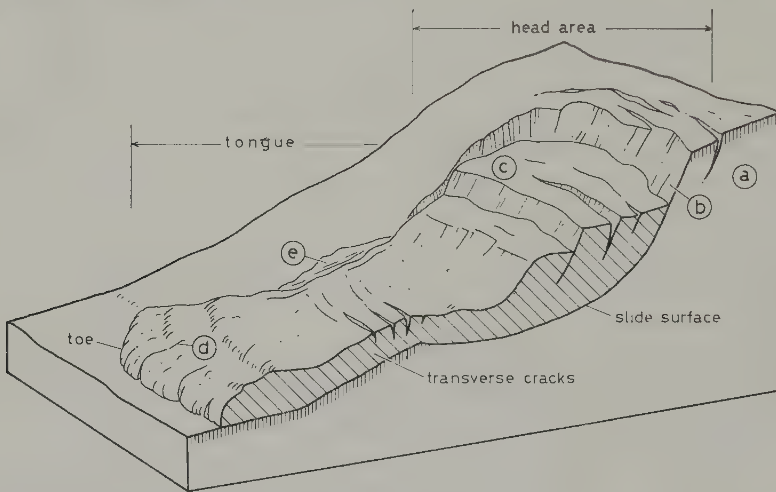


Fig. 5–26. Principal parts of a landslide with a cylindrical slide surface and characteristic cracks (after Varnes, modified); a — lunar cracks, b — head scarp, c — transverse cracks, d — radial cracks, e — lateral ridges.

Rotational slides in clayey rocks vary in size; those in artificial cuttings are relatively small, whereas those occurring on high river banks or on sea coasts may attain enormous dimensions. Occasionally many millions of cubic metres may be set in motion.

Under particular conditions rotational slides may develop into *multiple rotational slides* by retrogressive extension of the slope failure. As a result, two or more subsidiary slides are produced, their slip surfaces passing into a common slide surface. Each of the subsequent slides rotates backwards but the overall movement approaches movements of the translational type as the number of the failures increases. The form of multiple rotational slides differs depending on the material involved. A common variety of such slides develops in valleys eroded in stiff clays capped by jointed competent rock, which retards the degradation of the scarp (Hutchinson 1968).

The steep concave slopes of river valleys consisting of clayey rocks may be affected by a whole series of slumps, which have developed a group of interlocking alcoves enlarging thus the slide laterally. The slumped masses often reach into river channels, and the removal of this material by flood waters causes a new disturbance of the slope equilibrium.

A section through the landslide at Březno in the Ohře-river valley (Fig. 5–27) may serve as an illustration of a simple rotational slide. The slide is situated on the northern slope of an elevation which towers above the flat relief; a hard layer of baked claystones on the top has protected the underlying Cretaceous marls from denudation. Lateral erosion caused by the meandering of the river carved a steep slope almost 40 m high and finally disturbed its stability, the shear resistance of

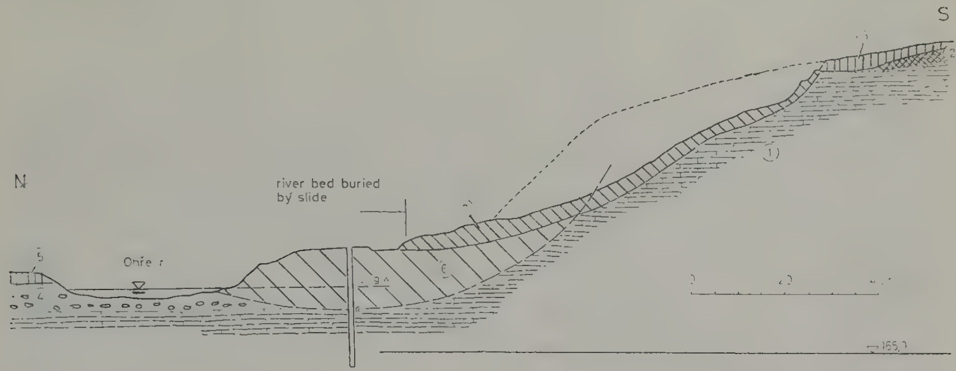


Fig. 5–27. Geological section of the landslide at Březno near Postoloprty (after Pašek, modified): 1 — Turonian sandy marls, 2 — baked Neogene clay, 3 — loess loam, 4 — sandy gravel, 5 — alluvial loam, 6 — old slide (marls), 7 — younger slide mass.

the marls being overcome. The marls slipped down along a cylindrical slide surface and at the foot of the slope a large block of Cretaceous marlstones was squeezed out into the river. They are clearly visible on the river bank as beds dipping at 10–15° into the slope; this angle of inclination indicates the course of the slide surface, because away from the landslide the Cretaceous beds lie almost horizontally.

The temporary stability is also disturbed by the lateral erosion of the river which carries away the front margin of the slide. The course taken by the river indicates that the landslide at one time partly blocked the ancient Ohře channel, shifting it about 40 m towards the north.

In this stretch of the Ohře river there is a series of similar but smaller slumps, their kettle-shaped scars being clearly visible on the banks. The bulges of the slipped material have not, however, been preserved, the material having been carried away by floods (Fig. 5–28).

Sites of high slumping incidence are the valleys of the Volga, Moskva, Dnieper and other major rivers in the Soviet Union. The slumping is most extensive in the middle and lower reaches of the Volga valley from the town of Gorki as far as Volgograd, especially on the steep western banks. The movements involve not only young superficial deposits (mainly loess and loess loams) but also the bedrock. In the steep banks, which have been carved by lateral erosion, subhorizontally bedded clayey and sandy sediments of the Permian, Jurassic, Cretaceous and Tertiary periods are exposed. The sliding phenomena occur in various forms and show different stages of development. Slumping along cylindrical surfaces occurs mostly where the slopes are formed of Jurassic and Cretaceous beds. The conditions that led to the slide

Fig. 5-28. Kettle-shaped scars of old landslide on the banks of the river Ohře (photograph by Záruba).

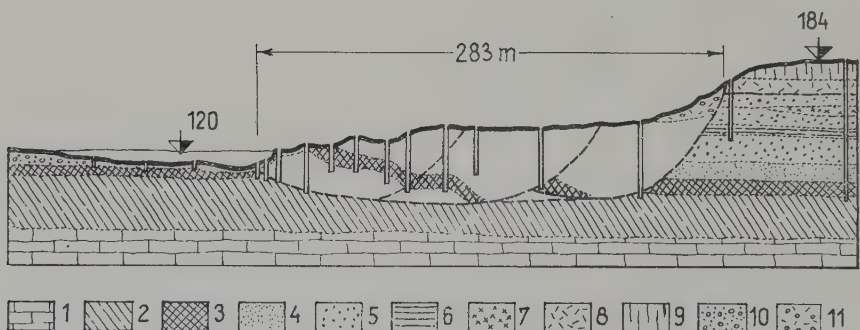


Fig. 5-29. Cross-section of a landslide of the Volga type, after I. V. Popov; 1 — Carboniferous limestones, 2—3 — Jurassic clays, 4 — sands, 5 — Neocomian sands and sandstones, 6 — clays, 7 — sands (Aptain), 8 — submorainic sands, 9 — moraine, 10 — alluvium, 11 — slope debris.

movements in the Volga valley were repeated several times in geological history, as is suggested by fossil slides buried by younger river deposits. Figure 5—29 shows a cross section of a landslide of the Volga type, after Popov (1951). The cylindrical slide surface developed in the Jurassic clays, and the slope movements also affected the overlying Cretaceous sands and clays together with glacial deposits.

Popov (1951) noted that the sliding of clays occurred when lateral erosion brought about a release of pressure on the clay, leading in turn to a greater absorption of water, and a decrease in the strength of the clay. In some places the original loading of the clays was reduced owing to a deepening of the valley of about 100 m so that the unloading amounted to 1.5 – 1.9 MPa. This accounts for the susceptibility of the clays to swelling.

The landslides in the Volga valley are up to several hundreds of metres long and several dozens of metres deep. On the concave banks one slide depression lies beside another and the remnants of the original slope intervening between the slumps form striking crests which remain standing for a long time. Two factors contribute to their stability; the loading of their toes by downslipped masses and the drainage provided by deep marginal cracks in the head areas. On slopes of greater height there may occur several slides overlying one another. Landslides with similar characteristics in the valleys of the rivers Manych and Dnieper near Kiev have been described in detail (Rogozin 1958, 1971).

As an example of a multiple rotational slide is often cited the landslide near Folkestone in southern England (Fig. 5 – 30). In the steep shore, upper Cretaceous rocks overlie the Gault clays. During the last two centuries a number of large land-

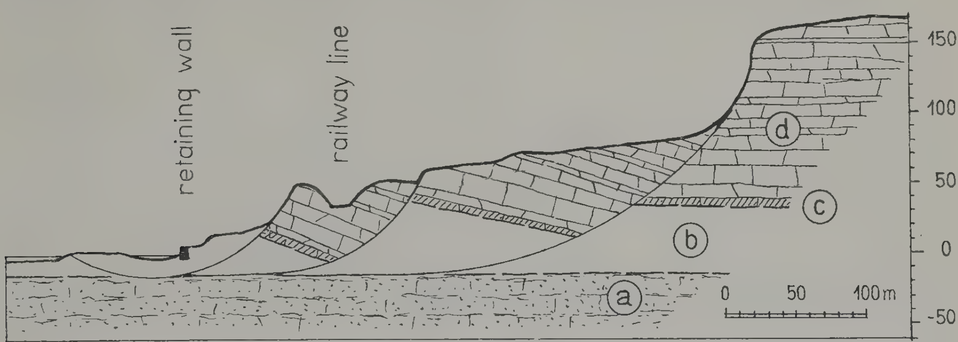


Fig. 5-30. Landslide on the seashore near Folkestone in England. a — glauconitic sandstones, b — Gault clays, c — glauconitic marls, d — sandy marlstone (Ward 1945).

slides interrupted the Dover-Folkestone railway on several occasions (Fig. 5 – 31). The cause of these landslides was marine abrasion of clays at sea level, which resulted in the release of load on the foot of the steep shore and, consequently, in its slumping along a curved slide surface. The stability of the shoreline was decreased by the hydrostatic pressure of water in the fissures and joints traversing the Cretaceous rocks. Tests on samples taken from borings have revealed that the Gault clays near the slide surface possess only a fraction of the strength of the undisturbed clays. As

Fig. 5–31. Railway line near Folkestone threatened by sliding (photograph by Záruba).

mentioned above, this reduction in strength can be attributed to the unloading of clay and the seepage of water into it, in this case from the glauconitic rocks.

In every new landslide near Folkestone, clays at the toe were squeezed out and heaved up into a ridge that rose from the sea as an island; gradually the ridge was removed by abrasion until it disappeared completely. It was clear that if the erosion of squezed-out clays were prevented, the danger of sliding would be diminished, because the weight of the clay would contribute to the stability of the slope. Thus, thick retaining walls and groynes were built along the shore at right angles to the

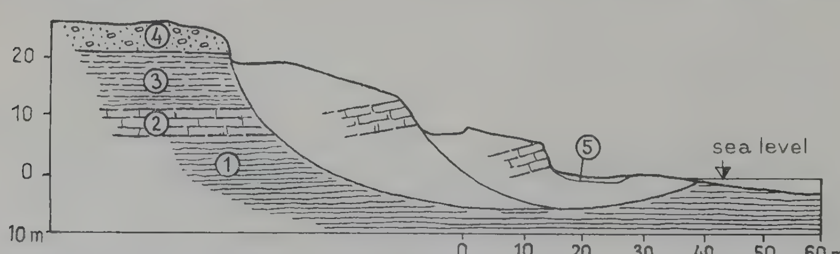


Fig. 5–32. Slump on the coast of the Isle of Wight. Cylindrical slip surfaces developed in soft clays of Oligocene age; 1 — clays and sands of Osborn Beds, 2—3 — limestones and marls of Bembridge Series, 4 — sandy gravel, 5 — young littoral sediments (Skempton 1946).

shoreline to protect it from abrasion and encourage the deposition of coastal sediments. As well as this, a drainage gallery about 250 m long was constructed in order to lower the ground-water level in the disturbed area (Legget 1962, Toms 1946).

Another coastal slump on the Isle of Wight in southern England is shown in Fig. 5–32. In this case, the movement was provoked by wave erosion and by a gradual weakening of the Oligocene fissured clay. On the sea coast, unconsolidated grey clays were overlaid by solid limestones which in turn were covered by sandy gravels. The cylindrical slide surface down which the mass slipped had a radius of about 90 m (Skempton 1946). On the shore, grey clays were raised up and in the head area two large blocks subsided and tilted to the slope. The slip of the two blocks temporarily restored a situation of balance and the slope became more stable. However, unless the clay ridge is protected against the surf and wave erosion, the stability will sooner or later be lost again.

5.2.2 Landslides along composite slide surfaces

In large landslides in clayey rocks a new slide surface generally develops in the upper part of the slide, but in the lower part the slide surface develops along a stratum of lower strength situated in an appropriate position. These movements are transitional in type between slides occurring along predetermined surfaces, and slumps.

A large landslide of this kind has been examined in the Váh river valley (Slovakia) near Súčany, where a power plant was built. There the river Váh flows along the southern foot of the Malá Fatra Mountains, in a broad valley cut in Neogene marls and silts with interbeds of friable sandstone and conglomerate. West of Súčany, marls and silts crop out in a steep concave bank, 400 m long and up to 24 m high (Fig. 5–33). This concave bank is the front of a slide, the head scarp of which is 900 m from the right bank of the river. The surface of the slide area is partly covered by an ancient alluvial cone. The deeply dissected ground surface is suggestive of recurrent

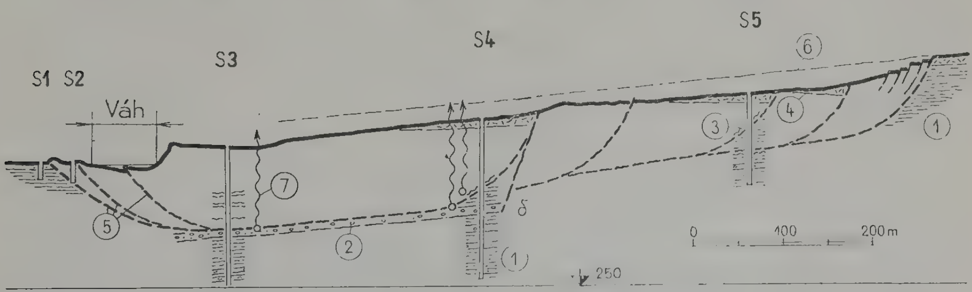


Fig. 5–33. Section of a deep landslide with a compound slide surface near Súčany at the foot of the Malá Fatra Mts., Slovakia; 1 — Neogene marls and sandy marlstones, 2 — Neogene sandy gravel, 3 — kneaded layers, 4 — granite debris, 5 — partial slip surfaces, 6 — original ground surface, 7 — confined ground-water in Neogene gravel (Záruba and Mencl 1958).

sliding processes in the geological past. An outcrop of the slide surface has been established on the left bank of the Váh, 60 m from the river channel. Neogene sediments have been raised several metres above the Holocene floodplain along this slide surface, and the bent willow trunks indicate that the slide has been in motion until recently. An interesting feature of this landslide is that the Váh flowed over the lower part of the slide, undermining its toe, while the slide surface crops out on the opposite bank (Fig. 5—34). River erosion thus acted in the opposite direction to the landslide

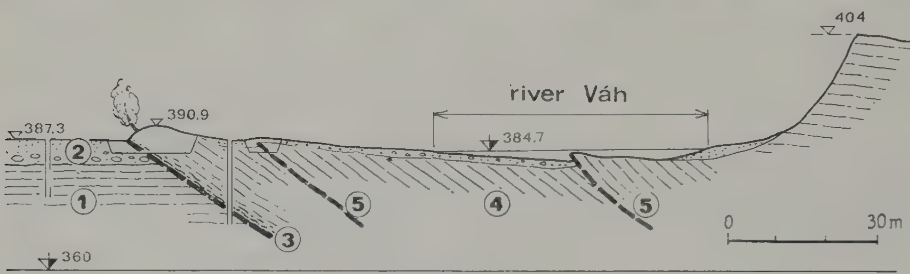


Fig. 5-34. Section of the front part of the Súčany landslide; 1 — Neogene marls in their original position, 2 — floodplain sand and gravel, 3 — kneaded layer squeezed out above the flood plain, 4 — raised Neogene marls eroded by the Váh, 5 — partial slip surfaces (Záruba and Mencl 1958).

movement. As the movement was relatively slow and the river erosion was strong, the two factors were almost in equilibrium. In the longitudinal profile of the river bed, a distinct knickpoint exists at the site of the landslide. Such steps originate in stream channels as a result of selective erosion in hard rocks, but in this particular case it formed in soft marlstones which were continuously pushed into the river channel by the sliding movement.

The slide area was surveyed and the rate of movement was measured by geodetic methods (Fig. 6—11). During the period 1947—1957 the horizontal displacement averaged 8—10 cm per year. The shapes of slide surfaces and moulded zones were studied by means of test-pits and boreholes. The shear zone dips 34—40° into the slope on the left bank, and was found at a depth of 80 m on the right bank. The outcrop of a partial slide surface was discovered in the river bed. The volume of the mass involved in the movement is estimated to be 40 million m^3 (Záruba and Mencl 1958).

This landslide developed in relatively flat terrain as a result of lateral erosion of the river Váh, and local hydrogeological and tectonic conditions. Basal conglomerates contained an artesian water horizon at a pressure of 1.3 MPa, which brought about the uplift of the moving mass. The movements partly occurred along ancient fault planes, which predetermined the shape of the landslide.

Stability analysis has shown that the results of laboratory tests do not conform fully with the real situation. Thus, when a large mass is in motion and the slide surface runs deep under the land surface, the results of box shear tests performed on a small scale cannot be relied upon to give an accurate picture of the large-scale movement. In these tests only one clear-cut slide surface develops, but at great depths and thus also at high pressures (in the case studied up to 2.1 MPa), the movement produces thick remoulded zones, in which the structure of the mass is totally altered; the changes involve even the finest particles. This fact was reported by Denisov (1951) who found that pressures exceeding 1 MPa cause a disturbance of the internal structure of clay and a reduction in the angle of shear strength.

On account of the Súčany landslide the Krpelany-Súčany-Lipovec water scheme had to be redesigned. The diversion canal and the Váh channel were relocated so as to avoid any damage that might have been caused by the movement. In any case, the movement ceased as soon as the toe of the slide was no longer undermined by river erosion. The slope was stabilized by a loading fill (about 60,000 m^3) placed on the valley floor where the squeezed-out ridges of Neogene deposits indicated the outcrops of slide surfaces.

Slope movements along composite slide surfaces occur frequently and they develop either as slumps passing downslope into earthflows moving on a predisposed surface, or by the gradual headward caving-in of the scarp (i. e. by extension of the slide uphill). Landslides of the first type may form where young slope deposits rest on clayey beds. The slopes of the broad valley of the Dřevnice river (Moravia) are composed of Paleogene pelitic shales with sandstone interbeds. Relics of gravel terraces buried by Pleistocene clayey flood sediments are preserved locally in the lower part of the slope. The slopes have been levelled by the accumulation of younger, locally very thick slope deposits, predominantly of clayey-sandy debris. On the concave river banks the slope deposits are disturbed by deep slides, with cylindrically shaped slip surfaces in their upper sections, the lower sections following the bedding plane of a clay layer overlying the terrace; the slides end as earthflows on the valley floor. During floods, the slipped material is removed by the water and thus the temporary stability of the slope is again disturbed.

Fig. 5—35 shows a section through one of these landslides which was investigated and surveyed because of the construction of a highway across the slope. According

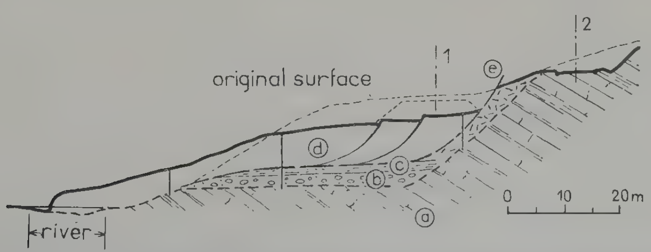


Fig. 5-35. Landslide of slope debris along a composite slide surface; a — argillaceous shales and sandstones, b — sandy gravel, c — clayey alluvium, d — slipped mass; 1 — intended highway, 2 — constructed highway.

to the original plan, the highway should have crossed the head scarp on a fill 3–4 m high, the weight of which would obviously result in renewed movement. The highway was therefore located higher up the slope beyond the slide area, where the bedrock cropped out and the beds dipped into the slope.

Complex landslide forms, usually occurring along composite slide surfaces, are caused by the reactivation of mass movements along ancient slide surfaces. The construction of the diversion canal for the Hričov-Mikšová water scheme in Slovakia disturbed the equilibrium of one side of the Váh valley, which has been at rest since the Pleistocene. The canal feeding the power-plant near Mikšová was directed through a cutting up to 20 m deep. The first sign of movement appeared in September 1962, when an arcuate crack about 150 m long and initially 15–20 cm wide developed above the edge of the cutting. The slope above the cutting is gently inclined and passes into a wide terrace plain formed of Pleistocene gravel and covered by loess loams. At the site of sliding, the bedrock is made up of Tertiary conglomerates with intercalated pelitic shales.

During the Pliocene and Pleistocene, the Váh river carved out a broad valley flanked with terraces of sandy gravel covered with loess-loam. The base of the largest terrace in the section studied is at an altitude of 332–334 m above sea-level (37–39 m above the present Váh level). In the excavation for the canal and in the borings, the Váh gravels were encountered at several levels between altitudes of 322 and 330 m. In our opinion these levels represent one and the same terrace that had been lowered by sliding movements. The section through the slide exposed in the excavation for a drainage trench also bears out this explanation.

At the foot of the slope there is a relic of an extensive gravel aggradation with its base approximately at the present river level. Calcareous silty alluvium filling the abandoned channel of the Váh is preserved on the surface of this terrace. The aggradation which yielded rich interglacial fauna, was preserved under a thick loess-loam cover. Fig. 5–36 illustrates the progressive development of the relief on the right river bank and the origin of the ancient landslide (Záruba and Ložek 1966b).

After aggradation of the terrace (b), intense river erosion deepened the valley by 46 m. The steep concave bank provided favourable conditions for the slide movement, which affected both the terrace gravel and Tertiary deposits. The large slide was probably not a single-phase event but rather the result of a series of events. Subsequently, aggradation filled the valley with sandy gravel up to the 313–316 m level. During the following interglacial substage, limnic sediments were deposited at the foot of the slope in the abandoned channel of the Váh. The accumulation of gravel contributed towards a complete stabilization of the landslide and the entire area was covered by loess loams. Subsequently the valley was gradually deepened to its present level and the flood plain sediments were deposited. During this long period the slope was quiescent and only when its foot was unloaded by excavation, did movement along the ancient slide planes recur. The ground water issuing from the terrace gravel and from the jointed conglomerates was an important contributing factor towards the resumption of movement. The slope was stabilized by means of drainage trenches and borings from which the water was pumped, but as the pumping is a relatively expensive operation, a drainage gallery was driven and the overlying rocks were drained into it by several oblique borings.

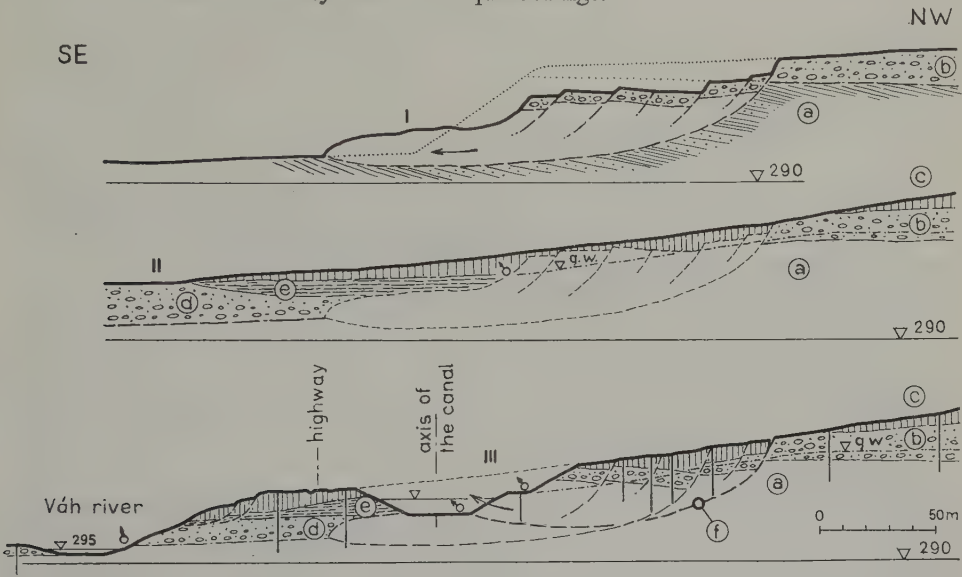


Fig. 5–36. The history of the Mikšová landslide on the bank of the Váh river. Strong valley erosion following aggradation of the terrace (b) gave rise to landslide (I) in Tertiary shales and conglomerates (a). Subsequent aggradation (d) was followed by the deposition of interglacial silts (e) and the covering of the slope by loess loam (c). The landslide became stable (II). The excavation for the diversion canal caused a renewal of movement on ancient slip surfaces (III); f — drainage gallery.

An illustrative example of a composite slide surface is provided by the landslide at the margin of the brown-coal basin near Karlovy Vary. The failure occurred to a depth of 30 m in a complex of volcanic rocks, consisting of basalt interstratified with basalt tuff and agglomerates. Fig. 5 – 37 shows that two lava flows separated by tuff beds were sheared off. The movement occurred on a well-defined slide surface, which was steep and slightly curved in the upper part and nearly horizontal lower down. From samples of tuff weathered to clay taken near the slide surface it was found to be of a high plasticity. According to the activity value (after Skempton) which was greater than 3 and after X-ray, and differential thermal analyses, the predominant clay mineral was identified as montmorillonite. The slope failure is thought to be Pleistocene in age since the material filling the head area above the sunken surface of the volcanics, i. e. stony basalt detritus and sandy loam, gives an impression of strong mechanical weathering (Záruba and Rybář 1970).

Large-scale fossil landslides of this type in the Nile valley in Upper Egypt were reported by Solle (1967); a thick complex of limestone and marly shale of Eocene age slid down along curved surfaces. Near the valley floor the slide surface runs within subhorizontal clayey shales rich in montmorillonite (Esna shales). The landslides occurred in some of the Pleistocene pluvials so that under present-day arid climatic conditions they have become a fossil phenomenon (Fig. 5–38).

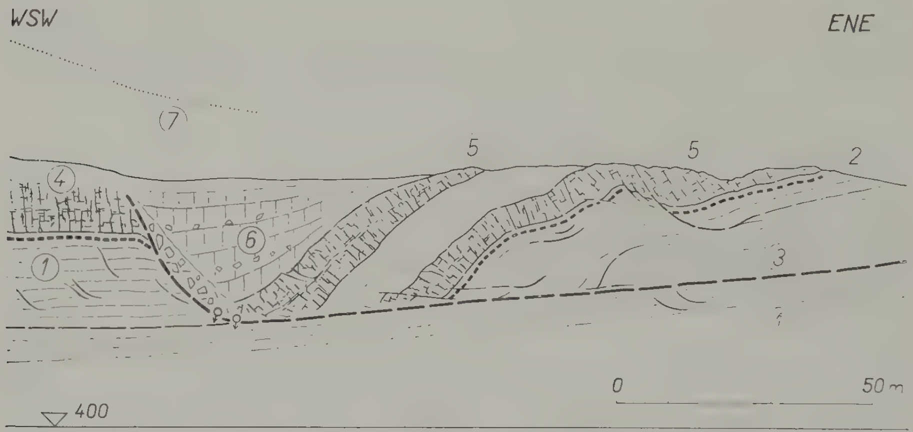


Fig. 5–37. Section through the Pleistocene landslide at the margin of the brown-coal basin near Karlovy Vary (Záruba and Rybář 1970); 1 — weathered clayey tuff, 2 — coaly clay, 3 — slip surface, 4 — basalt in original position, 5 — basalt flows deformed by slope movement, 6 — basalt debris and slope loam filling the head area, 7 — presumed original ground surface.

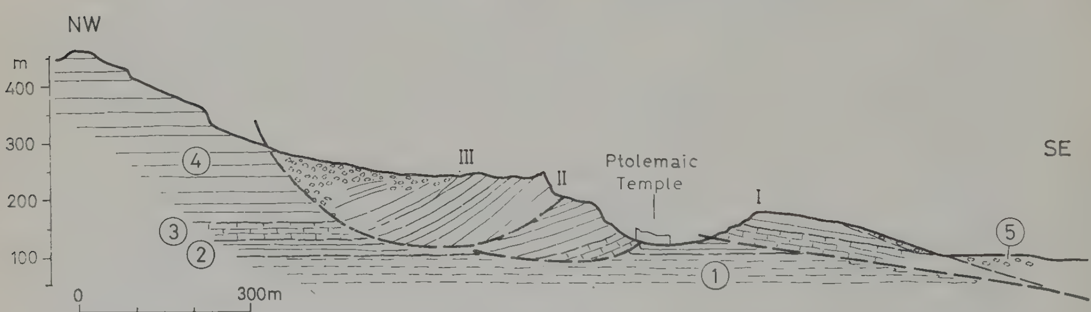


Fig. 5–38. Pleistocene rockslide in the valley of the Nile near Deir el Medine, Egypt (after Solle, 1967); 1 — Cretaceous chalk and shales, 2 — Eocene Lower Esna shales with montmorillonite, 3 — Upper Esna shales, 4 — Eocene limestone, 5 — sandy gravel.

5.2.3 Slope movements caused by the squeezing out of soft rocks

The form of slope deformations resulting from the squeezing-out of underlying soft rocks depends on local geological and topographical conditions. This group includes block slides, slope deformations caused by the squeezing-out of soft clays in the floors of erosion valleys or cuttings, and some failures of embankments caused by the small bearing strength of the substratum. The squeezing-out of soft rocks is a widely occurring natural process which because it proceeds extremely slowly, escapes attention.

This process takes the form of plastic deformation of the rocks along a system of partial slide surfaces. The deformation is generally associated with many minute displacements within the shear zone. The differential shifts do not connect to form a uniform slide surface, and this gives the movement the character of plastic deformation; the sliding is so slow that the process may in many cases be classified as creep. The instability of the slope is perceptible only over a long time interval, during which the continuous minute deformations reach measurable values.

In the later phases of the movement, the minor partial slide surfaces may join to form an enduring surface along which, conditions permitting, an abrupt movement takes place (e. g. a block slide). The squeezing-out of soft rocks can be reconstructed in the laboratory by loading a layer of asphalt with a flat plate. At first, no deformation is observable, but after some time the plate sinks into the asphalt which behaves as a dense viscous fluid and spreads outwards. The process is very slow and is produced by a load that is far below the limit of the compression strength as measured by current test methods.

(a) *Block slides* develop in places where the soft clay beds underlie jointed solid rocks forming high sheer walls. The marginal blocks of the overlying rocks, separated by joints, gradually sink into the soft substratum squeezing it out and move downwards. The tension produced in the marginal parts of the solid rock tears it apart and gaping fissures arise. As a rule, the lower part of the block moves outward and the upper surface of it inclines into the slope.

The edges of the Cretaceous Plateau (e. g. in the environs of Prague and in the Turnov area) with their steep walls of solid sandstone resting on soft marlstones provide favourable conditions for block slides. Fig. 5—39 shows the Cretaceous beds

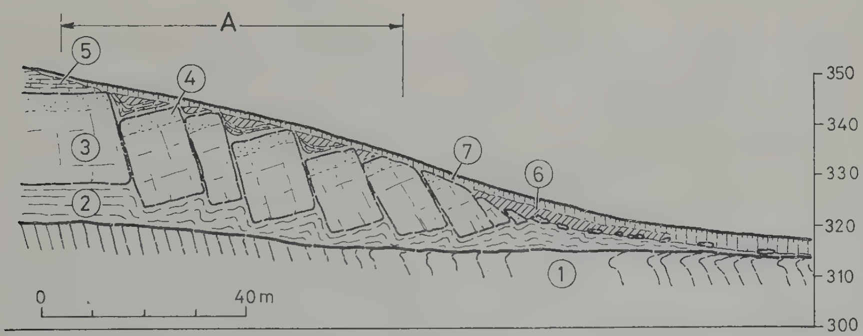


Fig. 5-39. Pleistocene block slide of Cretaceous beds in the Motol valley in Prague; 1 — weathered Ordovician shales, 2 — Cenomanian clays, 3 — Cenomanian sandstone, 4 — glauconitic sandstone, 5 — Turonian sandy marls, 6 — older loess blanket, 7 — young loess blanket. A — area shown in photograph, Fig. 5—40.

exposed in an abandoned quarry on the northern slope of the Motol valley in Prague. Several large sandstone blocks have sunk into soft Cenomanian clays and shifted downslope. The bedding of the sandstone blocks dips 14–16° into the slope and in the steps above there are relics of the Turonian sandy marlstones which were dragged out during the sinking of the sandstone blocks (Fig. 5–40). On the floor of the quarry,

Fig. 5–40. Block slide at the margin of the Cretaceous Plateau on the outskirts of Prague. Large blocks of sandstones have sunk into soft underlying clays and have moved downslope (photograph by Záruba).

bulges of stiff dark-grey Cenomanian clays squeezed out by the heavy blocks can be seen. Cretaceous clay and sandstone boulders have also been found in the test pits lower down the slope, having been redeposited by periglacial solifluction. The movement even affected the weathered Ordovician shales underlying the Cretaceous clays. In this case, the slide is a fossil phenomenon because the slope is covered by two loess-loam layers separated by the relics of a fossil soil. The older loess-loam with its abundant flat fragments of sandy marlstone is preserved at the foot of the slope and levels out the steps above the inclined sandstone blocks. The younger loess loam forms an almost continuous cover and is evidently younger than the slope movements described above. When the blocks sank into the clays, these must have been softer than they are today, and their softening may reasonably be attributed to thawing of the perenially frozen ground in the Pleistocene (Záruba 1943).

Block slides occur frequently in the České středohoří Mts., where the margins of many basalt sheets are affected. A large block slide occurred near Bystřany in the

Teplice area (Fencl and Záruba 1956). The elevation situated to the north-east of Bystřany is formed by a basalt sheet lying on tuffs and marlstones, and at the edge of the slope a head scarp is visible below which the sheet is broken into several blocks. These blocks sank into plastic marlstones, which on being squeezed out, moved down the slope along with the basalt blocks (Fig. 5—41). Analogous slope movements have

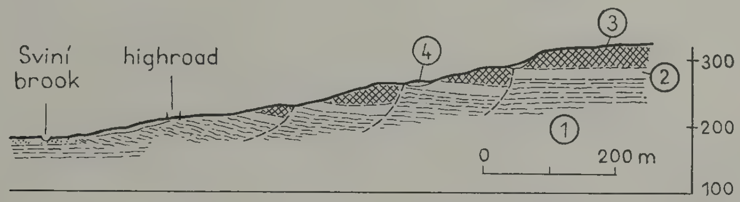


Fig. 5–41. Section of a block slide near Bystřany in northern Bohemia; 1 — Senonian marls, 2 — basalt tuff, 3 — basalt sheet, 4 — loess (Fencl and Záruba 1956).

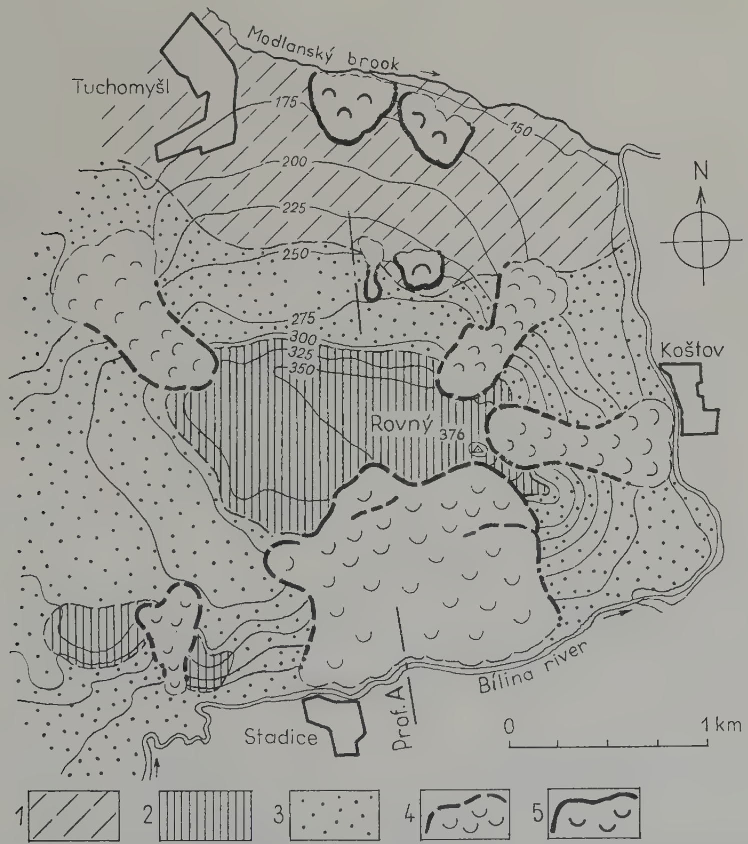


Fig. 5–42. Geological map of landslides on Rovný Hill near Stadice (northern Bohemia); 1 — Neogene clay, 2 — basalt sheet, 3 — tuffs and tuffites, 4 — fossil landslides, 5 — recent landslides (after Kleček).

been established in a basalt quarry near Obrnice (Fig. 1–7) and in many other localities in the České středohoří Mts.

Thus, for instance, the slopes of Rovný Hill to the north of Stadice in the Bílina river valley, are scarred by landslides of various ages and sizes (Fig. 5–42). The largest of these is on the southern slope where tuffs, clays and diatomaceous clays are capped by a basalt sheet approximately 70 m thick. Several basalt blocks have shifted down the slope, one of them up to 200 m long having tilted 18° upslope to form a conspicuous edge (Fig. 5–43). The head scarp is almost 1.4 km long and is

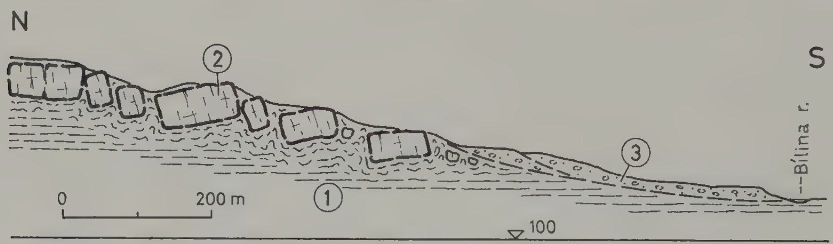


Fig. 5–43. Section of fossil block slide on the Rovný Hill (along line A in Fig. 5–42); 1 — tuffs and tuffites, 2 — basalt blocks, 3 — landslide material.

partly covered with bouldery basalt scree. Slide movements began in the Pleistocene but fresh deformations and cracks observable on the surface of the boulder field indicate that the slope is not yet at rest (Pašek and Demek 1969).

In Slovakia extensive block slides occur on the periphery of Tertiary effusive sheets covering pelitic rocks of Palaeogene and Neogene age. Sheets of andesite, basalt, liparite and their agglomerates are dissected at the margins into blocks which gradually sink into the soft substratum and move down into the valley. Blocky and bouldery scree which has accumulated below the steep marginal walls has overloaded the upper parts of the slope and brought about deep landslides in the underlying argillaceous rocks. Many of these block slides date from the Pleistocene; at present they are mostly at rest but are capable of being reactivated by exceptional climatic conditions or human interference (Nemčok 1964).

Large-scale landslides of this type have been observed in the Slovenské rudohorie Mts. on the periphery of the Handlová coal-basin (Fig. 5–14), and at Kordíky near Banská Bystrica. The marginal slopes of the Slanské vrchy Hills (East Slovakia) are affected by deep block slides near Podhradí, Slanec, and other localities (Malgot 1977).

Disastrous block slides have occurred in the city of Algiers; the section in Fig. 5–44 shows the geological structure of the built-up elevation of St. Raphael. This is composed of densely jointed Lithothamnion limestones and a steep wall is formed where the marginal blocks break off and sink into Miocene marls overlying sandstones and conglomerates. The downslipping of the limestone blocks is triggered by high rainfall; the movements causing the greatest damage occurred in 1829, 1845, 1942, and 1943. The most recent disaster stimulated inter-disciplinary research that has provided an example of successful collaboration between geologists, geographers, geochemists and specialists in soil mechanics (Agard 1948, Drouhin et al. 1948).

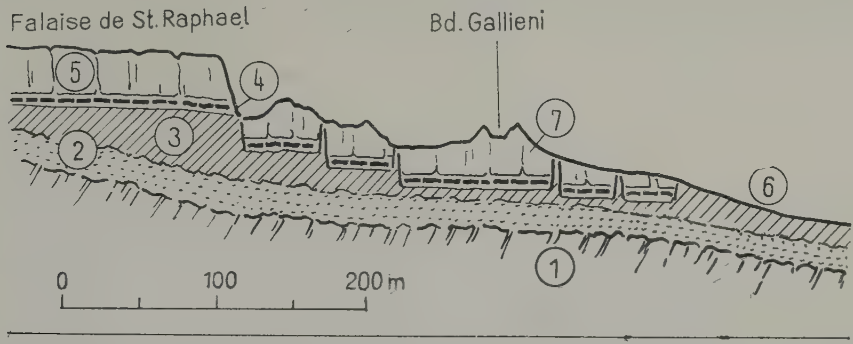


Fig. 5-44. Section of the block slide in the city of Algiers; 1 — crystalline schists, 2 — sandstones and conglomerates (Miocene), 3 — marls, 4 — glauconitic marls, 5 — Lithothamnion limestones. 6 — slipped marls, 7 — sunken limestone blocks. (From Agard, 1948).

The results of the investigation have shown that the marls under the marginal blocks have been weakened so that their strength is insufficient to bear the load of the overlying limestones. The low strength of the marls is explained by the chemical activity of glauconite which is present in a layer of clays at the base of the limestones. Potassium compounds released by weathering of the glauconite replace calcareous salts in the percolating water and thus significantly increase its pH. The alkalinized water is capable of dispersing the fine particles of the marls into suspension which is then carried away. The water content of the marls increases and the rock assumes the character of a viscous substance. Water seeping into the jointed limestones contributes to their weight, the joints widen and the cohesion of the rock decreases until the blocks break off and start to move.

Extensive block slides occur in the valleys of the rivers Angara and Ilima (Baikal region, Siberia). Most of them have been studied in the course of investigations for the Bratsk water schemes. Fig. 5—45 shows a cross-section of a typical Angara block slide. The slopes there are formed of Ordovician quartzites and sandstones lying on Cambrian claystones, into which the marginal quartzite blocks are sinking (Palshin et al. 1963, Trzhtsinskii 1964).

Numerous block slides are known on the Black Sea coast in Bulgaria. An interesting and relatively little known one is the Tauk Liman slide (Fig. 5—46) in the northern part of the Bulgarian coast (Kamenov et al. 1973, Pašek and Košťák 1977). The slide area is about 6 km long, 500 m broad and the depth of disturbance ranges from 40 to 50 m. The coastal slope is made up of two rock complexes differing fundamentally in their physical properties. The upper part of the slope consists of solid Miocene limestones 30—40 m in thickness, which overlie Oligocene clays more than 100 m thick. The latter were raised tectonically high above the sea level in pre-Sarmatian times. The water content of the clays is just above the limit of plasticity, and their strength parameters are 20 to 50 times lower than those of the overlying limestones. The Young's modulus for the clays is on the average by three orders of magnitude smaller than that for the limestones.

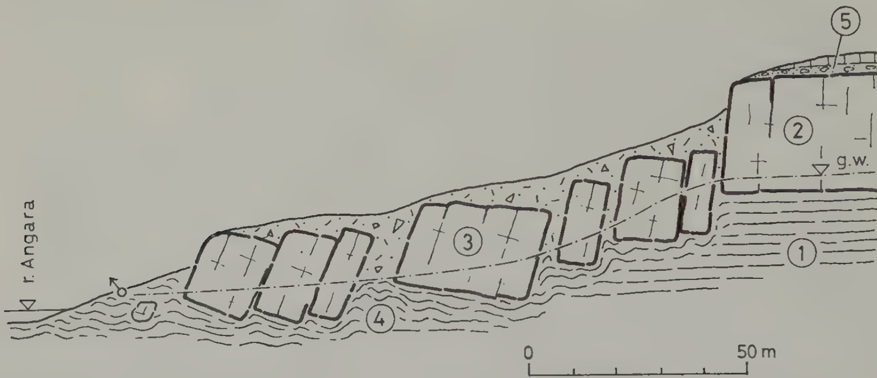


Fig. 5–45. Block slide on the left bank of the river Angara near the Bratsk Dam, Siberia; 1 — Upper Cambrian claystones, 2 — Ordovician sandstones, 3 — sunken sandstone blocks, 4 — weathered claystones disturbed by plastic deformation, 5 — terrace gravel (after Palshin et al., 1964).

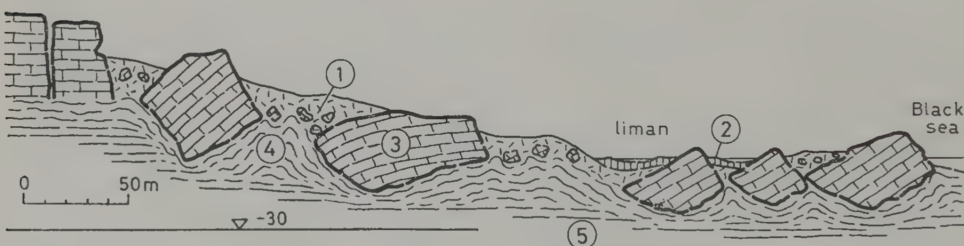


Fig. 5–46. Section through the block slide on the Black Sea coast (Tauk Limman, Bulgaria); 1 — slope debris, 2 — lacustrine sediments, 3 — blocks of Miocene limestone, 4 — kneaded clays, 5 — undisturbed Oligocene clays (after Kamenov et al., 1973).

The limestone blocks separated by longitudinal fissures are sinking gradually into the underlying clays which are being squeezed towards the sea. This process has occurred repeatedly over a considerable time scale and seven steps have been formed between the seashore and the edge of the plateau. The development of block slides has been aided by earthquakes. The remnants of walls which were built by the Thracians some 3000 years ago and which are preserved in the frontal part of the landslide, indicate the great age and relatively low velocity of the slope movements. Recent surveying has shown that the rate of movement is very low. Confidence in the considerable degree of stability of the slope has enabled a seaside resort to be built on this tract of the coast.

An analogous type of landslide has been described by Benson (1940) from New Zealand's sea coast. In the Dunedin district, sheets of basalt lava with tuffs and agglomerates lie on Eocene sands and glauconitic claystones underlain by Upper Cretaceous sandy claystones. The basalt sheet has been broken into a number of blocks which slide seawards. The construction of a railway line along the coast that should have crossed this slide area had been interrupted. The movements are recurrent and the tongue of downslipped material is so rapidly washed away by wave and current action that the coast recedes several metres annually.

(b) Slope movements caused by the squeezing out of soft rocks in the valley floor. In some regions the squeezing out of soft rocks in the valley floor is a widespread phenomenon which may occur on such a scale as to cause serious economic problems. These phenomena have been described in the iron ore opencast mines in the surroundings of Northampton in Central England by Hollingworth et al. (1944), who introduced the terms *bulging* for the squeezing up of clayey rocks in the valley floor and *cambering* for the subsidence of marginal blocks. Relatively deep valleys have been carved in solid Jurassic limestones and pelitic shales (Lower Oolite) and on the valley floors soft Lias clays have been laid bare. Although the complex is nearly horizontal, on the slopes and under the floor the beds are largely deformed. The beds of solid limestones and sandstones bearing iron ore are inclined into the sides of the valley, whereas the clays in the valley floor are squeezed upwards. The marginal blocks are at a considerably lower level compared with the original level which can be seen in the surrounding elevated ground. The surface limestone and sandstone

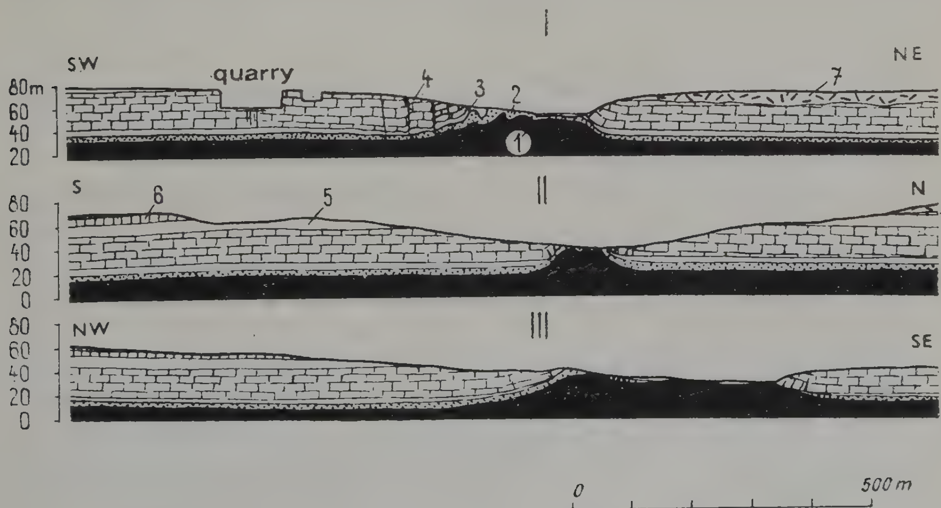


Fig. 5–47. Bulging of Lias clay in a valley floor in Lincolnshire, England; I, II — section of the Bytham brook valley, III — section of the Guash r. valley; 1 — Lias clay, 2 — Northampton Sandstone, 3 — Upper Estuarian Clay, 4 — Lincolnshire Limestone, 5 — clay, 6 — Main Oolite Shale, 7 — glacial sediments (Hollingworth et al. 1944).

beds affected by slope movements are disrupted by open fissures which run approximately parallel to the contour lines and which sometimes contain relics of younger rocks that have fallen into them.

As the marginal blocks sink on the slopes, the Lias clays in the valley floor are squeezed upwards. Although these deformations affect the rocks to a depth of several dozens of metres, in the geological sense they are a superficial phenomenon; they have developed only in the uppermost beds of clay beneath the valley floor, disappearing at the foot of the slope both downwards and into the slope; behind the slope the beds preserve the usual, almost horizontal attitude (Fig. 5–47(. In the initial stage, the bulging up manifests as a slight anticlinal bend of the beds; with increasing deformation the clays bend into minute folds and at the foot of the slope faults parallel to the valley course may even form.

The forces responsible for heaving up the clay result from the difference between the loading of the clays in the valley floor and the loading under the slopes. The deformation can be interpreted in terms of the squeezing-up of a plastic substance from a region of greater load into a less loaded area. These surficial deformations of the rocks and accompanying phenomena call for a lot of attention in engineering projects; they may be initiated by the excavation of cuttings wherever solid rocks overlie beds of soft material. Disturbances of this kind are particularly troublesome in excavations for dam foundations, as was found during the construction of several dams in southern England.

A major valley bulge has been recorded by Kellaway (1972) in the cut-off trench of the Darwell dam in Sussex (Fig. 5–48). The cross-section shows a broad belt of

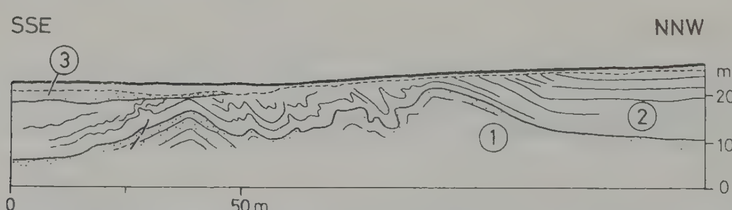


Fig. 5–48. Valley bulges in the cut-off trench of the Darwell Dam near Mountfield (England); 1 — sandstone, 2 — clays and shales, 3 — sand and gravel (after Kellaway, 1972).

crumpled strata of Cretaceous clays and sands probably representing only the basal portion of the original structure. The upper part was presumably removed by erosion before the deposition of the Pleistocene sediments now present at the surface.

Cambering and bulging processes have also been studied in the course of investigations for the Empingham Dam on the river Gwash in Central England (Horswill and Horton 1976). The river valley is deepened in the Upper Liassic, predominantly argillaceous sediments. Boreholes and excavations have shown that the clays under the valley floor are severely disturbed and crumpled. As a result of micropalaeontological examination, it was possible to divide the Upper Lias complex into five zones and thus recognize and correlate the deformations and changes in the thickness of the zones. Fig. 5–49 illustrates a valley bulge forming a complex anticlinal structure in the valley floor; the overlying rocks (Northampton Sands) in the valley sides were drawn out by cambering processes. It has also been found in boreholes and excavations that the fabric of clay to a depth of about 20 m was disturbed to such a degree that the clay assumed a brecciated structure. Fragments of solid claystones are

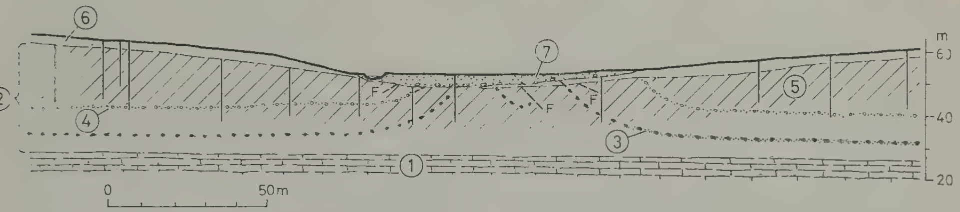


Fig. 5–49. Valley bulging exposed by excavation for the Empingham Dam in England; 1 — marlstone, 2 — Upper Lias clays, 3 — pisolite bed, 4 — Ammonite nodule bed, 5 — brecciated clays, 6 — ironstone head, 7 — gravel and alluvium, F — faults (after Horswill and Horton, 1976).

enclosed in a matrix of softer clay, a feature which is probably caused by *permafrost.* The bulged zone in this area is some 100 m broad, the beds beyond it being only slightly upwarped. The underlying Marlstone Rock Beds have remained practically undistorted and unaffected by bulging.

In Czechoslovakia, the foundation of the Žermanice Dam in the Ostrava area provided a good opportunity for the study of bulging deformations. The valley of the river Lucina was carved in marly shales of Lower Cretaceous age which are penetrated by teschenite sills. The shales are soft, marly, thinly laminated and are intercalated with thin seams of calcareous sandstone. When dry, the shales disintegrate into small flat fragments and when wetted they slake. The teschenite (a basic igneous rock) is mostly coarse-crystalline and very firm when fresh, but has decomposed to a considerable depth along several joint systems. Upon contact with the teschenite, the superjacent and subjacent shales are contact-metamorphosed to hard hornfelsic rock showing sheet jointing. The cross-section of the valley (Fig. 5–50) constructed from data collected during the surveying of outcrops and from core borings, shows

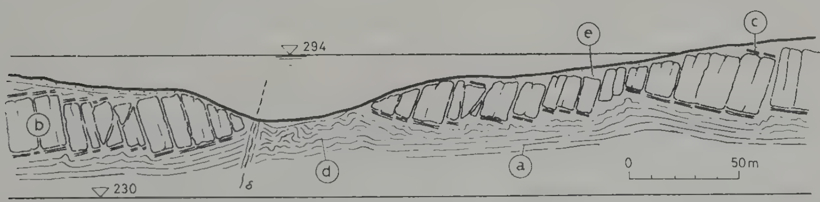


Fig. 5–50. Squeezing-out of marly shales in the valley bottom of the Lucina river near Ostrava; a — marly shales (Cretaceous), b — teschenite, c — contact metamorphosed slates, d — disturbed shale beds in the valley bottom, e — loess loam (Záruba 1956).

that the teschenite body is broken into several blocks by a system of faults running approximately parallel to the valley. The soft shales squeezed out by the heavy teschenite blocks moved towards the stream which gradually carried them away. The angle of inclination of the sinking blocks is indicated by the relics of contact-metamorphosed slates above the teschenite blocks. The main deformations probably date from the Late Pleistocene because the steps between the blocks are filled with slope debris and loess loam.

The question arises as to whether this phenomenon dates from periglacial times or whether it could develop under present climatic conditions. Generally, the squeezing up of the substratum may occur wherever soft rocks become unloaded over a limited area either by natural or artificial interference, the unloading giving rise to stresses. These stresses, even if they do not overcome the shear resistance of the plastic beds, may result in deformations provided that they persist for a sufficiently long time.

The periglacial freezing and thawing of pelitic rocks largely facilitated their deformations. Pelitic shales were enriched in water down to the depth limit of the frost action, which froze into ice laminae in the joints and bedding planes. As ice behaves as a plastic substance, the movement might have taken place even at the stage of freezing. Moreover, the bulk density of the shales was considerably reduced within the frozen layer. According to our experience, it is very probable that the perenially frozen ground reached a depth of at least 20 m and up to 40 m in this area. The difference between the bulk density of heavy teschenite blocks which could absorb relatively little water and that of the frozen shales loosened by ice laminae was considerable, the ratio of respective bulk densities being about 2.8 : 2. This difference was a contributory factor in the squeezing out of the shales, similarly as the low bulk density of salt contributes to the bulging of salt deposit. The movement was probably accelerated further during the thawing of the perenially frozen ground when the melt waters disturbed the consistency of the shales, so that their complexes became softer and more plastic than they had been when frozen and more so than they are at present (Záruba 1956).

The squeezing-out of soft rocks in the floors of erosion valleys is also known in Rumania, where this phenomenon is referred to as "*valley anticlines*" (Voitesti 1938). Rumanian geologists have studied the mechanism of the pressing up of salt deposits and have described various structural deformations occurring in these deposits as a result of being unloaded by the erosive activity of a young stream. Plastic deformation is a distinctive feature of salt deposits, but they also occur in the Tertiary pelitic sediments in the basins of the rivers Oltul, Tarnava Mare, Arges and others. Anticlinal bends occur below the valley floors, and landslides disturb the adjacent slopes although the beds are nearly horizontal away from the valley (Ilie 1955, Záruba 1958b). The concidence of the orientation of the "valley anticline" axes with that of the drainage network provides evidence that these phenomena are the result of valley erosion. At the mouths of tributaries, the branching of the anticlines is observable.

The landslides on the slopes of the Leninske Gory in the Moskva river-valley also involve the squeezing out of clays (Churinov 1957). Jurassic clays have been squeezed out by the load of Lower Cretaceous rocks and heaped into a ridge 20 m high on the valley floor. The same author mentions that the thickness of the zone affected by plastic deformations is about 20 m, as indicated by test borings and laboratory results. The downslipped blocks of Cretaceous rocks are buried by sandy terrace and morainic material, which points to the Late Glacial date of the movements.

5.3 Slides of solid rocks

Slope movements in solid rocks are produced either by natural agents during the deepening of valleys or by human interference. In Central Europe the naturally caused recent rockslides are relatively scarce, being restricted for the most part to mountainous areas. However, detailed investigation has revealed fossil slide deformations of many valley slopes, dating predominantly from Pleistocene times; in most cases these slides would not have been found had it not been for excavation works for dams or other structures.

Large-scale rockslides on mountain slopes are numerous in the Alps, these slides having occurred particularly after the retreat of glaciers in the Late Pleistocene. Many rockslides and rockfalls are described in detail by Heim in his book "Geologie der Schweiz" (1919 – 1922) and in several monographs. In the last of these (1932) Heim also presented a very elaborate classification of the Alpine rockslides, distinguishing twenty types of movement on the basis of rock type, the mode of rock deposition, the velocity of movement and a number of other factors.

The deformations of mountain slopes in the Alps have been studied in connection with the construction of large dams (Ampferer 1939; Kieslinger 1960; Clar and Weiss 1965; Zischinsky 1966; etc.). The two last-named authors base their work on the papers of Sander (1948), who used the term "*Hangtektonik*" to refer to slope deformations caused by gravity, at the same time pointing out the difficulty of differentiating them from true tectonic phenomena.

Rockslides in the Western Hemisphere have recently been discussed by Voight et al. (1978) in a comprehensive volume which gives an account of a great variety of rockslides and their disastrous effects in inhabited areas.

5.3.1. Rockslides along pre-existing surfaces

The rockslides are generally set in motion where the planes of separation (bedding, joint or fault planes) dip downslope and their continuity is broken at the foot of the slope. In stratified rocks with even and smooth bedding planes, the dip of the beds is usually the maximum inclination at which the slope is permanently stable. If the beds are undercut by river erosion or by excavation, they remain in position only by the force of friction. The coefficient of friction increases with the roughness and unevenness of the bedding planes, but friction can be reduced by climatic factors, by the freezing and thawing of water, or by the hydrostatic pressure of water in the joints if the free outflow of water is impeded. The stability of a slope may also be disturbed by an increase in its angle as a result of tectonic uplift.

The sliding of solid rocks on bedding planes is typified by a rockslide which in 1872 dammed a river valley near Mladotice (northern Bohemia), forming a lake 700 m long. The slide occurred on the eastern slope of the 110 m deep valley and involved Carboniferous arkoses and sandstones. The rocks slid along a continuous claystone bed dipping towards the valley at 8 – 12° (Fig. 5 – 51). The rockslide was

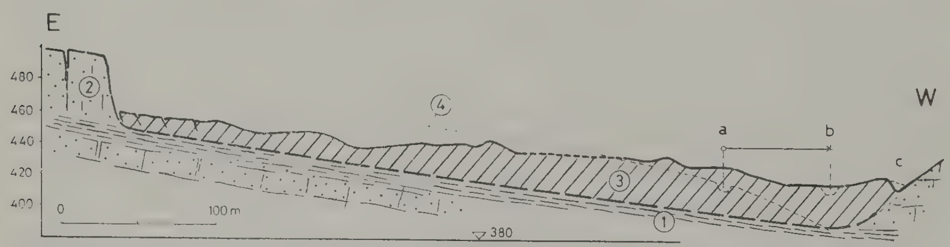


Fig. 5–51. Rockslide along a bedding plane near Mladotice in NW Bohemia; 1 — siltstone and claystone (Carboniferous), 2 — arkoses and sandstones, 3 — slide material, 4 — original slope surface, a — b — displaced railway cutting, c — relocated course of the stream (surveyed by Janský, 1976).

about 400 m wide, 600 m long and 20 – 30 m thick and the volume of the slipped material is estimated at about six million m^3. A steep head scarp in the upper part of the slope measures 30 m in height to this day. The slope below it is strongly undulated and is covered with bouldery debris (with boulders up to tens of m^3 in size).

The slope movement was triggered by heavy rainfall. However, the stability conditions of the slope may already have been disturbed to some extent by excavations for a railway cutting which were begun shortly before the rockslide. After the slope failure the railway line was relocated on the other side of the valley.

Rockslides on bedding planes or other surfaces of separation may reach disastrous proportions in mountain areas where there are great height differences and therefore rapid acceleration in the movements.

An instructive example of this type of slope failure is provided by the rockslide near Flims in the Graubünden Canton in Switzerland. With its volume of twelve cubic kilometres (after Heim 1932) it is the largest rockslide known to have occurred in Central Europe during the Pleistocene. The stability of the mountain slope, formed chiefly of Malm marly limestones dipping 7 – 12° towards the Rhine, was probably disturbed in the Riss/Würm Interglacial after the retreat of the valley glacier. The head scarp, particularly its eastern part which is known as Flimserstein and is about

1,000 m high, is clearly visible so that the original height and shape of the slope may be inferred from it. In the last phase of movement the moving mass attained a high velocity and rose 150 m up the opposite side of the valley (Figs. 5–52, 5–53).

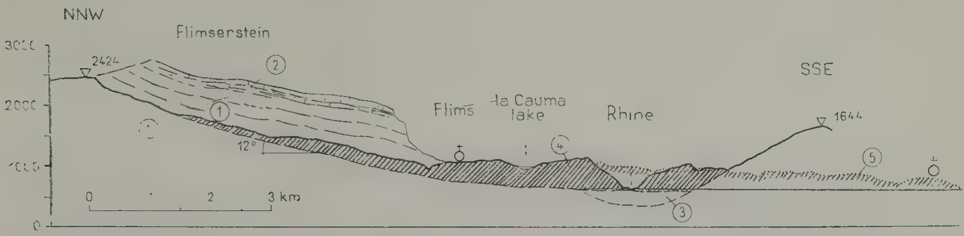


Fig. 5–52. Section of the rockslide near Flims in Switzerland; 1 — Jurassic limestones, 2 — marly limestones (Lower Cretaceous), 3 — old valley deposits of the Rhine, 4 — slipped Jurassic limestones, 5 — debris flow in the Rhine valley (after Heim 1932).

The Rhine valley was blocked over a distance of nearly 15 km and a lake formed upstream, the sedimentary and deltaic deposits of which are traceable for many kilometres. The river deepened its channel in the loose rocks by 400 m, so that the

Fig 5 - 53. Situation of the interglacial rockslide near Flims; a - outline of the head area, b — slipped Jurassic limestones (after Heim, 1932).

lake gradually dwindled away. The resulting slopes are very steep, consisting predominantly of limestone debris almost free of any vegetation cover (Fig. 5–54).

The slipped rocks initially covered an area of about 49 km^2. The height from the crown to the toe of the slide in the Rhine valley amounts to 2,000 m and the gradient is 8°. The area is at rest at present. Flims resort was built in the upper part of the slide area where some of the depressions of the hummocky surface are occupied by lakes.

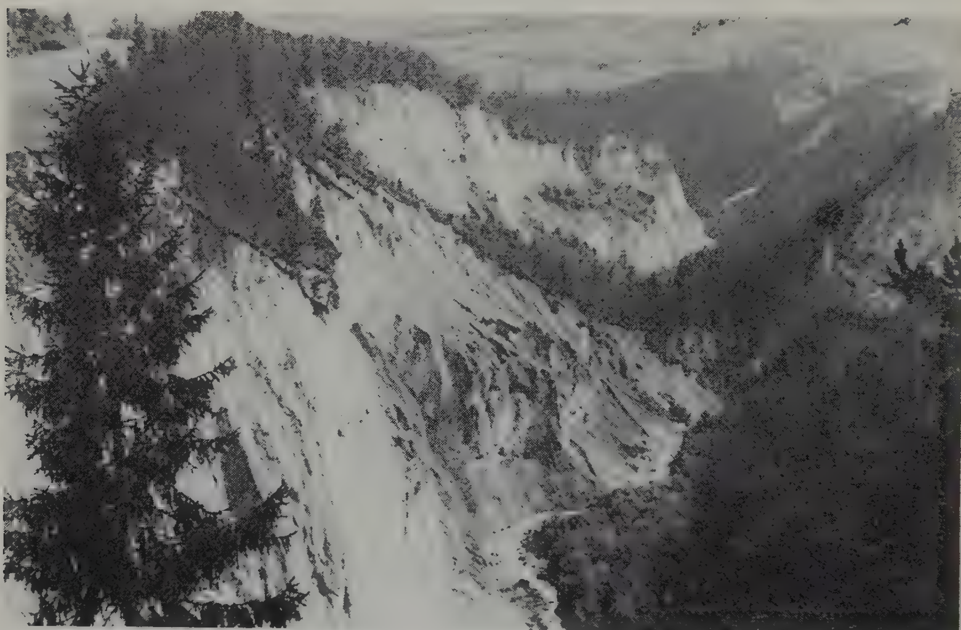


Fig. 5–54. The valley of a Rhine tributary cut in the limestone material of the interglacial rockslide near Flims in Switzerland (photograph by Záruba).

The disastrous slide that took place on the slope of Mt. Rossberg in Switzerland in 1806 buried the township of Goldau. A complex of Tertiary conglomerates slid along a bed of bituminous marls dipping 19–21° downslope (Fig. 1–2). The detached mass was 1,700–2,000 m long, 300 m broad and 60–100 m thick. Its total volume was estimated at 35–40 million m^3 (Heim 1919).

In this instance, several people were eye-witnesses to the movement. Some saved their lives by jumping over the marginal crack, but within one or two minutes the rock movement had reached a very high velocity destroying several villages in the valley (Heim 1932). As the slope had been stable throughout recorded history, the question of what had caused the disturbance was keenly discussed. Three main factors could have been responsible for initiating the rockslide: (a) orogenic forces may have brought about a gradual increase in the angle of slope until the coefficient of friction on the bedding plane underlying the loosened block was exceeded; (b) friction would have been further reduced as a result of the weathering of the rocks on the bedding plane; (c) the hydrostatic pressure of water in the bedding joints may have increased over a long period and caused uplifting of the loosened block. Terzaghi (1950) re-investigated the possible reasons for the catastrophe and arrived at the conclusion that the most plausible explanation lay in an increase in the inclination of the slope caused by tectonic movements, or a decrease in the bond between the slid block and the substratum as a consequence of weathering.

A slide of Jurassic limestones on a bedding plane overwhelmed the Vaiont reservoir in the Italian Alps. The enormous rock mass (more than 260 million m^3) filled the reservoir and put out of action one of the largest arch dams of the world. The water wave overflowing the dam destroyed the town of Longarone and devastated the valley of the river Piave downstream (see Chapter 10).

One of the largest rockslides that have occurred in North America in historical times is the Lower Gross Ventre slide in Wyoming, of 1925. A mass of more than 50 million m^3 of pelitic shales, sandstones and limestones slid down within an interval of three minutes. A bedding plane dipping about 20° towards the valley provided a slide surface. The masses blocked the valley and formed a temporary dam 80 m high and about 1 km long, ponding a lake 9 km long. After the spring downpours, the dam broke when the water overflowed the crown and the flood caused considerable damage in the valley (Legget 1962). This sliding event has recently been analysed in detail by Voight (1978); according to his interpretation it was the interplay of seismic activity and extensive precipitation that triggered the movement.

A number of extensive rockslides has been described by Cruden (1976) in the Canadian Rocky Mountains. The most serious of these was the Hope rockslide in the Cascade Mountains in 1965 (Mathews and Taggart 1978). The slipped rock mass of 47 million m^3 volume buried the British Columbia Highway over a distance of some 3 km (Fig. 5–55). The movement was so abrupt that the rock debris buried three vehicles moving along the road. The slide occurred on a mountain slope formed of green metavolcanics containing layers of intrusive felsite. Geological conditions of the site were conducive to slope movement, viz. the reduced shear strength along the intrusion layers, the schistosity and jointing of greenstone parallel to the slope surface, local serpentinization and the relatively rapid weathering of zeolite and carbonate fissure fillings. The rockslide is thought to have been triggered by earthquake events which also caused snow avalanches but it should be noted that the stability of the slope had not been disturbed by the earthquake of 1872 which was of much greater intensity. The 1965 earthquake set the rock mass into motion probably because of its small focal depth and suitable orientation of the horizontal component of the shock.

The coarse rock debris slid from a height of up to 1200 m above the valley floor and settled in the form of concentric ridges, pushing the muddy valley alluvium up the opposite side. The aerial photographs taken before 1965 show the definite scar

Fig. 5-55. Hope rockslide, Cascade Mts., Canada (courtesy of the National Research Council, Canada).

of an ancient slide that occurred after the retreat of the last continental ice-sheet. The recent slide involved part of the headwall of this prehistoric rockslide. Fissures on the mountain ridge above the head scarp suggest the possibility of further mass movement.

In Czechoslovakia, rockslides along bedding planes are common in the Carpathian flysch areas, where they occur not only on account of the properties of local rocks but also as a result of the topographical pattern (Nemčok 1964; Novosad 1966). In the Carpathians, the streams, owing to their high gradient, cut readily into the soft bedrock and the adjustment of the mountain slopes cannot keep pace with the rate of erosion. Consequently; the slopes become very steep, often steeper than the dip of the strata and where the dip is towards the valley, favourable conditions are created for the development of rockslides (Fig. 5–56).

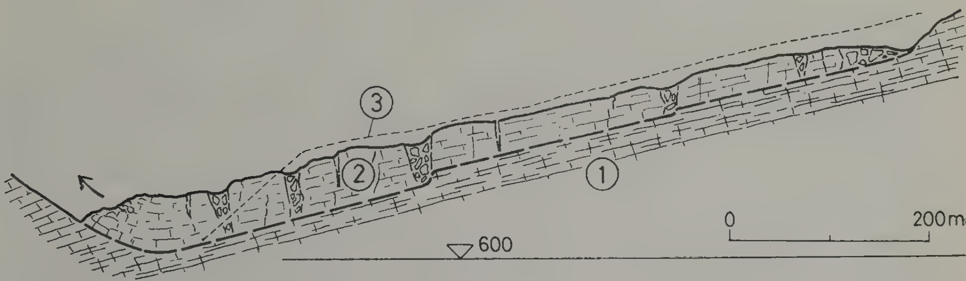


Fig. 5–56. Rockslide along a bedding plane; 1 — Cretaceous sandstones and claystones, 2 — displaced blocks, 3 — assumed shape of the valley before the slope failure (after Novosad, 1966).

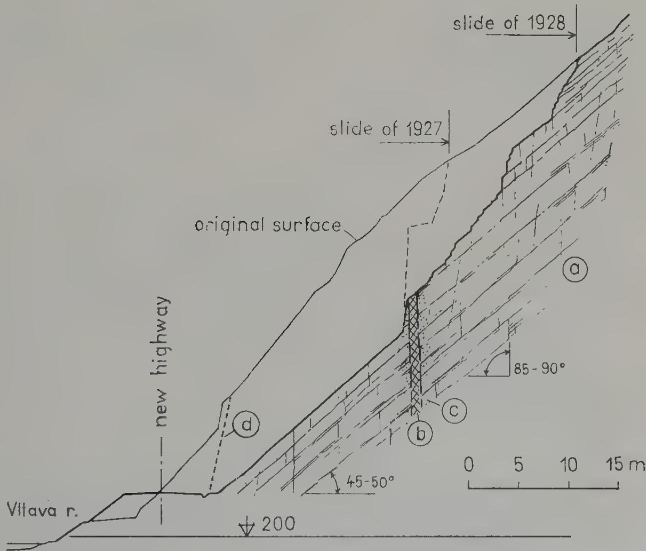


Fig. 5–57. Slide of Algonkian slates along bedding planes caused by a road cutting (d) in the Vltava valley south of Prague; a — Algonkian slates, b — diabase, c — contact metamorphosed slates (Záruba 1931).

Many rockslides of this type folow as a consequence of engineering projects such as building works and the mining of raw materials. The stability of the sheer rocky slopes of the Vltava valley was disturbed in several places by the construction of a highway. Figure 5—57 shows a section of one of these rockslides in which successive phases of the movement between 1926 and 1928 can be seen. The slates dip 45—50° downslope and are densely jointed. A temporary cessation of the movement occurred when a diabase dyke that pierces the shales along subvertical joints was laid bare. The diabase dyke is not very broad but combined with the surrounding contact-metamorphosed slates, it "reinforced" the slope for some time. The slope of the cutting was not designed in keeping with the geological structure of the area and the expected volume of excavation work was exceeded many times in some sections.

5.3.2 *Long-term deformations of mountain slopes (gravitational slides)*

Besides abrupt slides along predetermined surfaces, there are also slow long-term movements of rocks either on bedding planes or in rocks capable of plastic deformation (e. g. phyllites, mica-schists, chloritic schists). The latter deformations involve componental movements along planes of separation (planes of stratification, schistosity, foliation) without the development of a continuous slide surface.

The long-term loosening and sliding of rocks along moderately inclined bedding planes is manifested by a gradual opening of joints and fissures as a result of climatic effects (mainly the freezing of water in the joints), or is initiated by the release of residual stress in the rocks when a valley is deepened. In the galleries driven into the rock abutments of the dam on the Morávka river (in the Beskydy Mts.), it was evident that the loosening of the Godula Sandstones (which lie nearly horizontally) reached as far as 50 m from the slope surface. The movements on the slope of Mt. Lukšinec in Beskydy Mts. appear to be a slow continuous sliding of sandstone blocks on moderately dipping pelitic shales (Fig. 5—58).

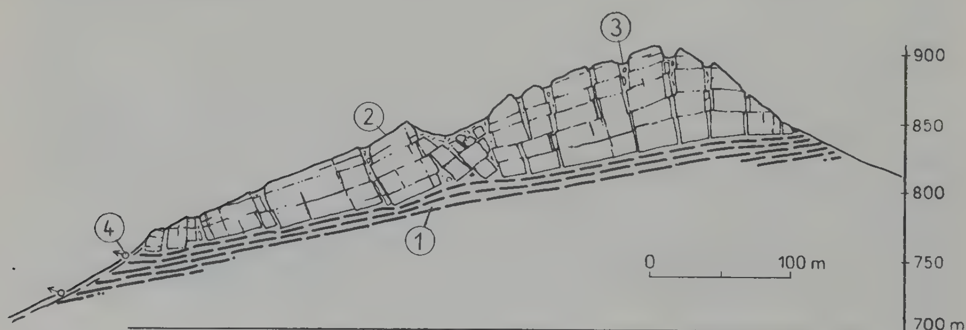


Fig. 5–58. Sliding of the Godula Sandstones on the slope of Lukšinec Hill in the Beskydy Mts. (modified from Novosad 1966); 1 — predominantly shales, 2 — sandstones, 3 — gullies, 4 — springs.

Long-term deformations of mountain slopes are referred to in the literature as "*gravitational slides*". They follow from tectonic movements which uplift sedimentary beds into a fold structure and thus upset the state of equilibrium. The uplifted complexes of beds which occasionally are of large extent, slide down the slope. The movements invariably occur on tectonic planes or inclined clay beds on which the coefficient of friction is low. Numerous gravitational slides of this kind have been described in the tectonically young mountain ranges such as the Alps, Apennines and Carpathians. Figure 5–59 shows an example of a gravitational slide from the Apennines in which a complex of solid Oligocene sandstones has slipped on a bed of variegated clays of Malm age (Giannini 1951).

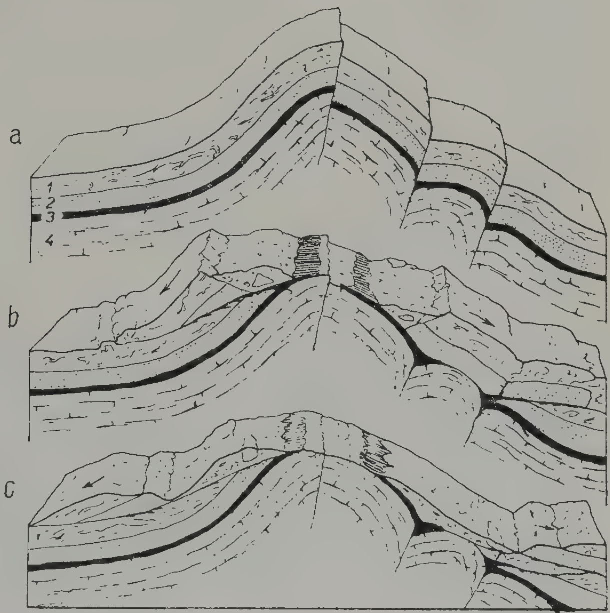


Fig. 5–59. Development of a gravitational slide (from Giannini); 1 — stratified clays, 2 — sandstones, 3 — red clays, 4 — siliceous limestones, a — upheaval of a faulted fold, b — gravitational slide of the apical part of the fold along the clay bed, c — final condition after partial denudation.

The deformations of mountain slopes in Iran described by Harrison (1936) as "*gravity collapse structures*" may be included in this group. They involve Mesozoic and Tertiary rocks uplifted by orogenic movements during the Tertiary. Their origin lies in the deep erosion of soft rocks in synclinal bends together with a gradual collapse of uplifted solid limestones which then slip down on gypsiferous marls (de Sitter 1956). Gravitational deformations extending deep into the mountain slopes have been described in the Alps by Ampferer (1939, 1940), who denoted them as "*Bergzerreißung*", and later by Stiny (1941, 1952). Kobayashi (1956) reported similar deformations producing constrictions in mountain valleys in Japan. Slow mass movements have been studied more recently by Zischinsky (1966, 1969), who recorded a series of "*Sackung*" events in the glacial valleys of the Austrian Alps. The long-term movements occur most frequently on slopes made up of phyllites, mica-schists and other crystalline schists which allow large-scale deformations to occur as the result of componental movements (*Teilbeweglichkeit* in the sense of Sander).

The section of a slope deformation from the area of Matrei-Glunzerberg in the eastern Tyrol is shown in Fig. 5–60. The 1,200 m high slope of mean gradient 24° is formed of phyllites and calcareous mica-schists. Although over the whole area

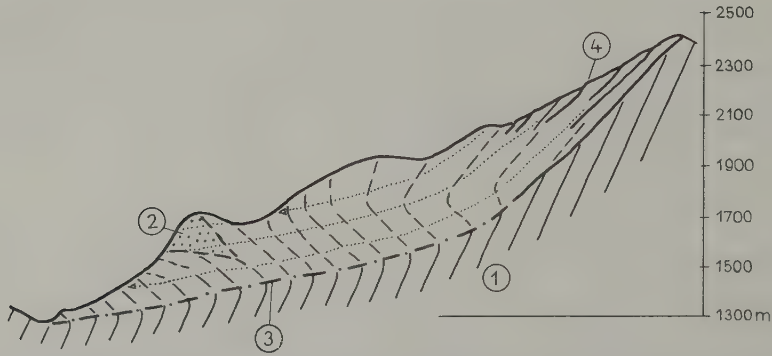


Fig. 5–60. Long-term deformation of the Matrei Glunzerberg slope in the eastern Tyrol (from Zischinsky, 1966); 1 — phyllites and paragneisses in original position, 2 — carbonate sediments, 3 — probable depth of slope deformation, 4 — componental slip surfaces.

the beds dip 50–60° to the south i. e. towards the valley, they are inclined at 30–60° to the north (into the slope) in the lower part of the slope (up to about 1,750 m above sea level). Zischinsky explains this change in terms of a partial rotation of the beds owing to plastic deformation within a broad zone. Consistent with the deformation of the whole complex of beds was the discovery of partial slide surfaces in the upper part of the slope, which indicate that there were differential slides within the body but do not constitute evidence of the slide of the entire rock complex. The above-mentioned author speaks of a "*continuous movement*" in view of the long-term course of the rock motion.

More than one hundred analogous deformations of mountain slopes have been recognized in the West Carpathians, particularly in the Tatra Mountains (Nemčok and Pašek 1969, Nemčok 1972). They have been found in slopes of both crystalline schists and granitoids and are topographically prominent particularly in the summit parts where the outcrops of slide surfaces form striking rock steps. Figure 5–61 shows a typical double-crest form of a ridge arising from slow gravitational movements.

· Creep deformations in crystalline schists (gneiss, micaschists, phyllite) show the characteristics of gravitational folding. As an example let us examine the profile of the Ráztoky ridge (Mahr and Nemčok 1977), which is buillt up predominantly of biotite and biotite-muscovite paragneisses (Fig. 5–62). The deep-seated deformation of the ridge is manifested by a set of scarps, the larger of which are up to 40 m high. The height of the slope above the valley floor is 812 m, its length 1,660 m and the

Fig. 5-61. View of the summit of Mt. Smrk (West Tatra Mts.). The scar demonstrates the movement of the ridge downwards to the left (photograph by Rybář).

average slope angle is 26°. In the upper and lower parts of the slope movement occurs on well-defined shear planes owing to the low normal stress and dilatant behaviour of the rock. In the central part, where there is a high normal stress, the deformation shows a contractant behaviour of the rock (Mencl 1968) and involves a wide zone. These deformations show a form of *S-like deep creep* in the sense of Ter-Stepanian (1965, 1974).

A very interesting example of "*Sackung*" in the Maratea Mts. in southern Italy has been described by Guerricchio and Melidoro (1979). The profile in Fig. 5–63 shows the geological structure of the slope formed of Triassic limestones and dolomites thrust on a complex of argillites and marls in flysch development (Flysch Nero). After the deepening of the valley the slope has been affected by intense deformations, which involved multiple sliding and gravitational phenomena. The latter display all characteristic features of a deep-reaching "Sackung" in an advanced stage of development. The Marathea Mts. is the site of young tectonic activity, which probably contributed

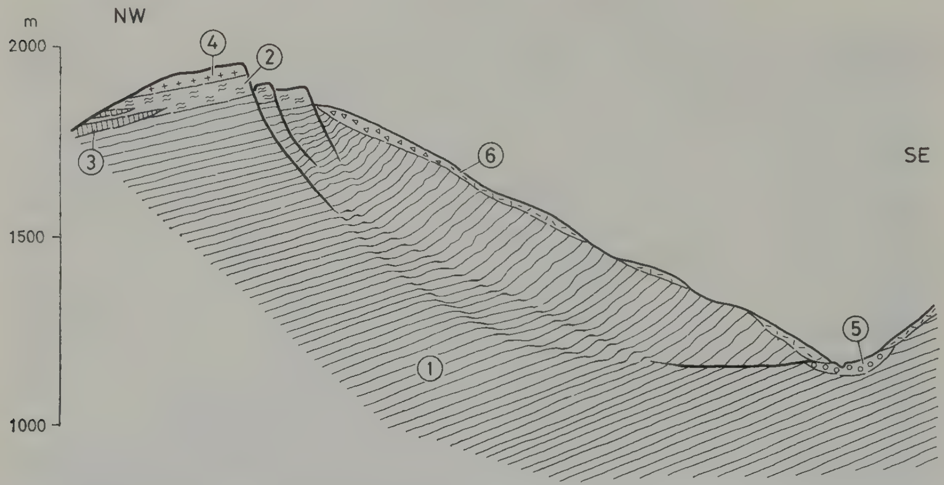


Fig. 5–62. Section through the Ráztoky mountain slope (West Tatra Mts.) showing gravitational deformation; 1 — biotite gneiss, 2 — migmatite, 3 — amphibolite, 4 — granite, 5 — moraine deposits, 6 — rock debris (after Nemčok, 1972).

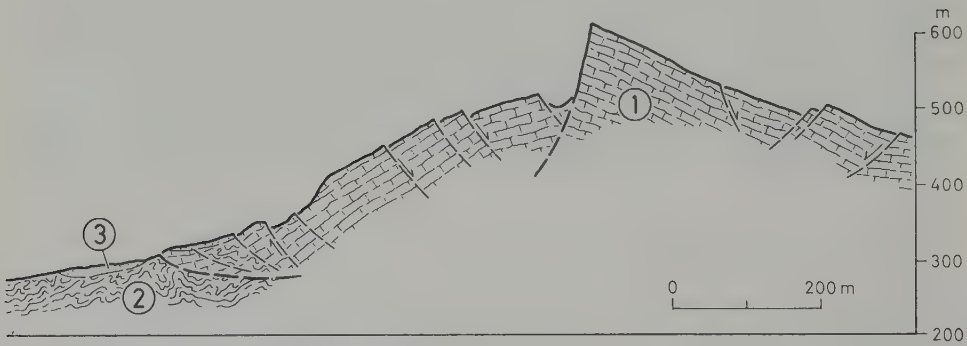


Fig. 5–63. Section of the gravitational deformation in the Maratea Mountains, Italy (after Guerricchio and Melidoro 1979); 1 — carbonate formation, Trias, 2 — Flysch, Cretaceous, 3 — talus.

to the formation of such clear-cut forms together with earthquakes which frequently occur in this part of Italy.

Gravitational movements and the spreading of steep-sided mountain ridges have recently been studied in various parts of the Rocky Mountains in the U. S. A. (Radbruch-Hall and Varnes 1976).

These deformations of mountain slopes need to be more widely recognized; they occur more frequently than is currently realized by most geologists and engineers. Many steep slopes loosened in this way are totally concealed by younger slope deposits. Timely recognition of these phenomena is of great importance in the construction of dams, especially those designed for pump-storage reservoirs in the high mountains.

5.3.3 Rockfalls

The term rockfall refers to abrupt movements of loosened blocks or complexes of solid rocks detached from steep rock walls. Rockfalls are distinguished by the very high velocity in free fall. Thus the fall of large rock masses detached from a position high up on a mountain wall may attain a velocity of up to 200 km per hour. The falling masses therefore possess a high kinetic energy and the rock debris scatters over a very large area at the foot of the wall, even where the ground is level. Some of the rock streams in the Alps and other high mountains have developed by this process.

Rockfalls are relatively common on steep mountain slopes, especially in valleys overdeepened by mountain glaciers. They are also abundant on rocky coasts of lakes and seas, as well as on concave banks of deep river valleys.

This category includes slope movements of widely varying dimensions ranging from the breaking-off and falling of isolated stones to the fall of enormous rock complexes. The chief causes of rockfalls are as follows:

(a) Gravity
(b) Jointing and tectonic fracturing of the rock
(c) Weather effects, wedging effects of freezing water in joints, the hydrostatic pressure of water in water-bearing fissures, other weathering processes, the pressure of plant roots, etc.
(d) Triggering factors such as earthquakes, the undercutting of slopes both by natural and artificial agents, and exceptionally lightning.

In the young mountain ranges – the Alps, Carpathians, Himalayas, Andes and Rocky Mountains – hundreds of rockfalls have been recorded. One of the largest rockfalls in the area of the Glarner Alps occurred on the eastern slopes of the Glärnisch-Guppen mountain range, to the south of the town of Glarus (Heim 1895). Marly limestone of the Cretaceous and Jurassic age, dipping at a moderate angle into the slope, fell into the river valley along near-vertical joints running transversely to the bedding planes. The valley of the Linth was filled over a distance of about 5 km and the debris rose up the opposite bank 230 m above the present valley floor (Fig. 5 – 64). The rock debris containing large limestone blocks covered an area exceeding 8 km^2, the volume of the rock mass being estimated at 800 million m^3. From the remnants of lacustrine sediments in the Linth valley it can be inferred that the lake created by the rockfall was about 70 m deep. As the river progressively cut its channel through the natural dam, the lake waned away. Morainic deposits were found both under and on top of the rock debris and therefore Oberholzer (1933) dated this rockfall as Late-Würmian.

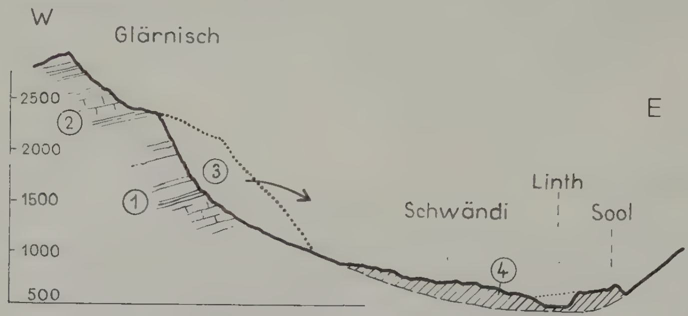


Fig. 5–64. Rockfall on the Glärnisch-Guppen mountain slope near Glarus in Switzerland (Heim 1895); 1 — Jurassic limestones, 2 — marly limestones and sandstones (Lower Cretaceous), 3 — head area, 4 — fallen block debris, mainly of Jurassic limestones.

Of the rockfalls that have occurred in the Alps the rockfall near Elm in Switzerland in 1881 is also worthy of mention; this fall (Fig. 5—65) has been studied in detail by Swiss geologists (Heim and Buss 1881, Heim 1932). The slope is composed of faulted shales and sandstones of the Flysch Zone which dip into the slope. The failure was

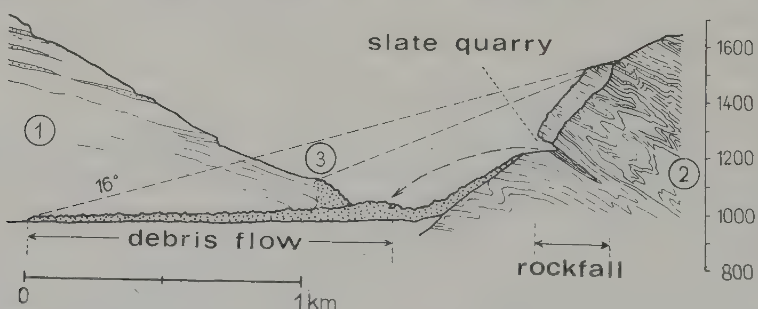


Fig. 5–65. The rockfall near Elm in Switzerland which was caused by the undercutting of a steep slope by a slate quarry (after Heim 1932); 1 — flysch sandstones and shales, 2 — Blattengrat Beds, 3 — debris pushed up the opposite slope.

triggered by undercutting of the foot of a steep slope in several slate quarries. The fall occurred in three phases with intervening intervals of 17 and 4 minutes, respectively. Within a few minutes more than 10 million m^3 of rock raised the level of a 90-hectare area of the valley by 10 to 20 m. Eighty-three houses were buried and 115 lives were taken. Heim stressed in his reports that the rock debris did not slide but rather flowed. From his description it seems that the debris moved as a *turbulent flow* of dispersed cohesionless grains in the sense of Bagnold 1956 (in Hsü 1978).

Rockfalls in the Carpathian Mountains are typified by the rockfall which disturbed the slope near the town of Georgheni in Rumania in 1828. The dislodged block of Dogger limestones dammed the valley creating the lake of Lacul Rosu which exists to this day.

In the Czechoslovak Carpathians, rockfalls have been recorded in the glacial valleys in the Tatra Mountains. The largest of these is probably the fall which came down from the slope of Mt. Slavkovský Štít into the valley of Studená Voda. A lake in the Blatná valley near Lubochňa owes its origin to a rockfall of dolomitic limestone (Záruba and Ložek 1966a).

The falling of loosened boulders from rock slopes is a much more common feature than the large rockfalls mentioned above, and occurs with particularly high frequency in the higher-latitude mountain regions of, for example, Scandinavia and Canada. The origin and frequency of rockfalls in the mountains of Norway have been studied by Rapp (1960), in particular the relationship between the number of rockfalls and climatic influences such as freezing and thawing and rainfall. For his analysis of the yearly periodicity of rockfalls he used records of rock falling along railway lines, mainly in crystalline and granitic sectors.

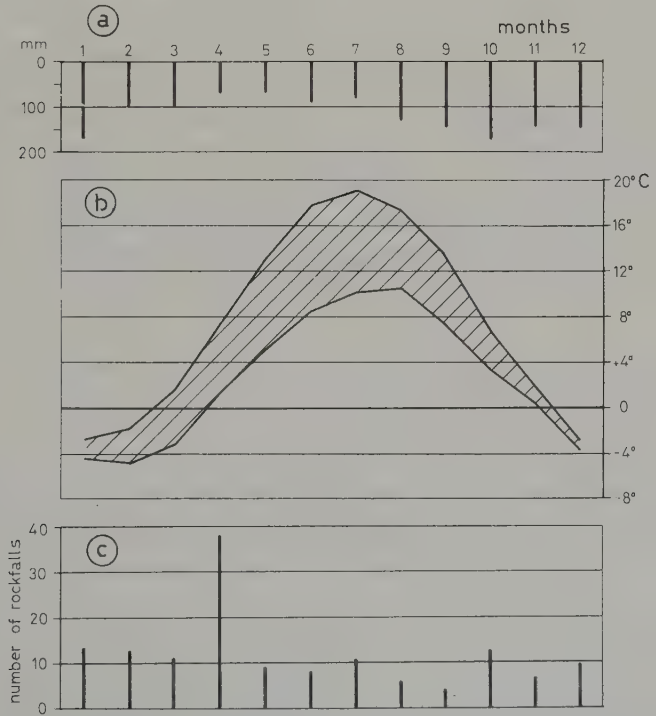


Fig. 5-66. The annual periodicity of rockfalls on three railway lines in western Norway between 1920 and 1958; a — precipitation, b — air temperature, c — number of rockfalls (after Rapp, 1960).

In Fig. 5–66 are plotted the monthly precipitation averages, the mean night minimum and day maximum air temperatures and the number of falls recorded on three railway lines in the Bergen area. The figure shows that the decisive factor controlling the frequency of rockfalls is the freeze-thaw effect: the rock split by the pressure of freezing water in the fissures disintegrates and during the spring thaw in April the loosened fragments and boulders fall away. This finding has been corroborated by observations along the railway line near Narwik in the north, where the maximum frequency of rockfalls is correlated with the delayed thawing in May and June.

Rockfalls are a matter of major concern in the Norwegian fjords since they often cause serious indirect damage to the coast (see Chapter 1). In the narrow fjords the large falling blocks give rise to swells several tens of metres high which devastate the inhabited banks (Jörstad 1968).

Many rockfalls are triggered by earthquakes, for example, the Friuli earthquake in the pre-Alpine area in 1976 gave rise to more than 250 rockfalls, which accounted for most of the mass movements that occurred at the time (Fig. 2–6). Boulders and blocks of various dimensions crashed from the rock walls formed of Triassic limestones, dolomites and flysch sediments, and were strewn around the foot of the slope and on the valley floor.

Many of the widespread rockfalls in the U.S.S.R. are also believed to be induced by earthquakes. Solonenko (1972) reports that the 1957 earthquake in the Baikal graben induced rockfalls in the Mujiskii khrebet at a distance of 220–230 km from the epicenter, the rockfalls occurring over an area of more than 150,000 km^2. He designates these phenomena as seismogravitational and mentions other examples, chiefly in Central Asia, from the Murgab river valley in the Pamir (1887, 1911) and in the Narym river valley (1946) which was blocked by rockfalls in several places. The earthquake in the southern part of the Tyan-Shan in 1949 also provoked rockfalls and landslides which caused greater damage than did the earthquake itself.

5.4 Specific types of slope movement

This group comprises slope movements which are specific to certain environments where they constitute an important factor in the modelling of the relief of the area and can be of great economic significance.

5.4.1 Solifluction

Solifluction is a slow downslope movement of surface material caused by freeze-thaw activity. In periglacial subarctic and high-mountain regions it is a significant slope-reducing agent. The deeply frozen ground thaws only to a small depth during the short summer. Since the lower layers remain frozen and prevent the percolation of melt water and precipitation water, the soil and other unsorted surface material becomes waterlogged and moves as a dense sludge even down quite gentle slopes.

Solifluction, however, is not confined only to areas with *perenially frozen ground.* Under the moderate climatic conditions of today it may occur on high-mountain slopes during the spring thaw, although to a far smaller degree as regards both the extent and depth of the flow.

The velocity of solifluction in mountain areas was observed and measured by Rapp (1960) in northern Norway. On slopes composed of mica-schist till and inclined at $15-25°$ the solifluction progressed at rates of up to 8 cm per year, exceptionally up to 30 cm per year. The layer in motion was about 50 cm thick.

The solifluction phenomena were important factors in the formation of the ground relief during the Pleistocene. The greater part of the scree covering present-day slopes was produced by Pleistocene solifluction. The well-compacted slope debris, consisting of small and coarser fragments of hard rocks embedded in loamy sandy or clayey matrix, is generally found in several overlying layers, which extend down slopes levelling out any depressions in the bedrock. The debris is usually covered by loess or slope loams on the surface of which a Holocene soil profile has developed. Since it is overgrown with vegetation, the slope debris is clearly a fossil product that came into being in the glacial periods in a climate similar to that of the polar regions at present.

In the environs of Prague, some Ordovician complexes of alternating jointed quartzites and shales have provided ideal conditions for solifluction. The densely jointed quartzites disintegrated when exposed to frost action into small, slow-weathering fragments, and the rapidly weathering shales provided the material for a lubricated filling of the interfragmental interstices. At present the debris of these Ordovician beds is very firm, difficult to dig in excavation works and resistant to climatic influences.

5.4.2 Sensitive clays

Landslides involving clay sediments of marine origin have special characteristics; after regression of the sea, the flat areas covered with clay sediments were rising, and extend up to about two hundred metres above sea-level at present. A gradual reduction of the salt content in the pore water of the soils has brought about a progressive decrease in their strength. As the height of the sediments above sea-level increased, there occurred a more rapid movement of the ground-water, which when percolating through the more permeable laminae osmotically impoverished the surrounding clays of salt. Water derived from precipitation had the same effect. Bjerrum (1955) reports that in one case the NaCl content of the pore water declined from $20-30\ \mathrm{g\ dm^{-3}}$ to $1.2-2.9\ \mathrm{g\ dm^{-3}}$. The reduction of salt concentration accompanies a decrease in the bond between clay particles and the bound water, and thus

Fig. 5-67. Air photograph of a landslide in quickclays near Skjelstadmark, Norway, in 1962 (courtesy of the Norwegian Geotechnical Institute).

there is also a decrease in the mechanical strength of the sediment. It is remarkable that the reduction in strength reaches a maximum towards the end of the process, when the salt concentration falls below 10 g dm^{-3}. It is characteristic that during this process the water content of the soil remains unchanged. The loss of strength leads to sliding of the clay, which under these circumstances behaves as a viscous fluid. The slides tend to grow in length and breadth and are very treacherous in that they may affect nearly level areas with gradients of less than 5 per cent. The movement is invariably very rapid. In Norway, these clays have been denoted by the term "quick-clays" (Reusch 1901), which is now the universally applied term for highly sensitive clays.

Figure 5–67 shows a landslide in quick-clays which occurred near Skjelstadmark north of Trondheim (Norway) in 1962. The movement was initiated by stream erosion which at first brought about a small slump on the bank. The sensitive clays that were thus exposed flowed rapidly away, filling an ancient valley to a height of 10 m. The landslide extended over a distance of 2.8 km, the head scarp was 12–15 m deep, and the movement involved about 2.1 million m^3 clay.

Slope failures in sensitive clays are common events in eastern Canada in the valleys of the St. Lawrence and Ottawa rivers; in recent years many instances have been studied and described in the geotechnical literature (Crawford and Eden 1964, 1967). One of the communities that suffered heavy damage from a landslide of this type was the town of Nicolet, Quebec. In November 1955 a part of the town "flowed out" within a period of several minutes, many buildings were demolished and three people were killed. The slide occurred on the right river bank though there were no indications of potential slope failure there. The movement is thought to have been provoked by construction works. The small initial slide uncovered a layer of sensitive clays. These then began to move and the slide extended retrogressively headward; the main scar reached far into the town and the displaced material flowed into the river and was carried downstream. In Fig. 5–68 are shown the sites where the debris

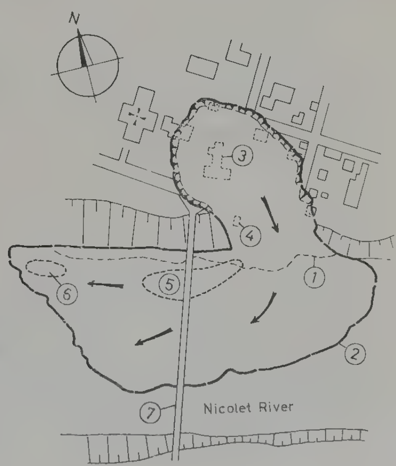


Fig. 5–68. Map showing the Nicolet landslide in Canada which occurred in Leda clay in 1955; 1 — river bank before landslide, 2 — limit of slipped clay, 3 — original position of the school building, 4 — original position of the garage. 5 — debris of school, 6 — debris of garage, 7 — bridge (after Crawford and Eden, 1964).

from the demolished school (5) and garage (6) were deposited. The bridge across the river Nicolet remained intact owing to the fact that the moving material was liquefied clay.

Sudden liquefaction of sensitive clays can also be caused by vibrations and shocks. The greatest damage caused to the town of Anchorage in Alaska by the earthquake of March 1964 was not inflicted by seismic shocks but rather by sliding movements triggered by the shocks. The town of Anchorage is located on a littoral plain made up of a bed of sand and gravel 20 m thick lying on a thick layer of clay. The earthquake tremors caused liquefaction of a bed of sensitive clay 7 – 10 m thick, which extends under the city area at sea level. The upper beds of stiff clay and the overlying sand and gravel literally "floated" seawards on the liquid clay. Towards the end of March the ground was frozen to a depth of several feet so that the moving rocks did not disintegrate, moving instead in large blocks. Many houses travelled several metres without suffering great damage although all subsurface service lines were torn. The blocks moved almost horizontally, the sensitive clay being squeezed into ridges in front of the slide. At the head of the slide deep grabens limited by antithetic fissures developed (Fig. 5 – 69).

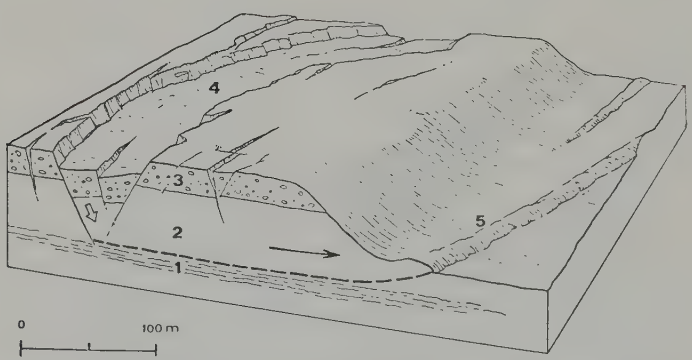


Fig. 5–69. Block diagram of a translational landslide caused by the liquefaction of sensitive clays during the Alaskan earthquake of 1964 (Hansen 1965); 1 — sensitive marine clays, 2 — stiff clays, 3 — sand and gravel, 4 — graben in the head area, 5 — squeezed-out clays.

5.4.3 Subaqueous slides

Subaqueous slides form by the slipping of unconsolidated sediments, especially clayey, silty and calcareous muds or saturated fine-sand alluvia, down the inclined shores. These slides occur on a small scale or a large scale ranging from a simple bending of the beds to total disruption of the structure and the creation of complicated slump formations and pseudoconglomerates with the fragments and galls of disturbed rocks.

Subaqueous slides occur in lakes and on seashores; favourable conditions for their initiation exist in deltaic deposits on fairly steep sea floors where deposition proceeds at a relatively high rate. The movement may be stimulated by seismic or other shocks. The slides involve either surface or subsurface beds which are set in motion as a result of the squeezing out of the underlying soft rocks.

Cases are known in which subaqueous slides of unconsolidated material pass into *turbidity currents*, so that the sediments loosened by the sliding movement become mixed with water and are deposited in more distant, deeper parts of the deposition basin.

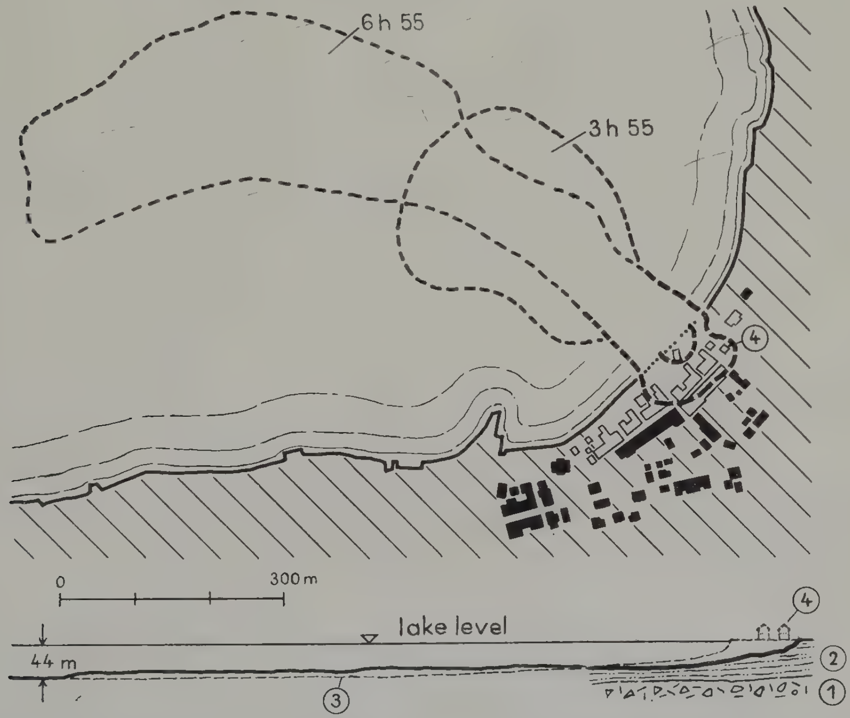


Fig. 5–70. Landslide on the shore of Lake Zug in Switzerland developed within three hours into a subaqueous slide 1,200 m long; 1 — morainic deposits, 2 — organic silts, 3 — original lake bottom, 4 — demolished houses (Heim 1908).

On the shore of Lake Zug in Switzerland, every decrease in the water level has resulted in some sliding of banks. The slide that was triggered by the driving of piles for a new embankment wall has been described in detail. Within three hours a subaqueous slide 1,200 m long and 200–250 m broad developed from a slip of the bank sloping at about 4% (Fig 5–70). The slide originated in sandy silts with an organic admixture (Heim 1908).

Systematic studies of submarine slides, carried out particularly with the use of seismic methods, have shown that these slides occur more frequently and are much larger than slides on the earth's surface (Moore 1978).

Their economic importance was realized when they caused damage to submarine telegraph lines. Recent developments in the extraction of oil from beneath the floor of shelf seas have also promoted interest in submarine slides which endanger the stability of sediments both on and close to, the continental slope.

A well known event is the cable breakage which occurred on the Grand Banks south of Newfoundland in November 1929, as the result of an earthquake. The shocks in the proximity of the epicenter instantly interrupted several cables and triggered a large slide which turned into a turbidity current causing the interruption of five more cables (Menard 1964).

A vertical section of the continental slope and ocean floor on which the cables were laid is shown in Fig. 5–71 together with the time sequence of cable breakage. The disturbance proceeded at an extraordinary speed and a number of hypotheses have been put forward to explain it. Kuenen (1952) ascribed the failure to a large landslide on the continental slope, which triggered a turbidity current with an initial velocity

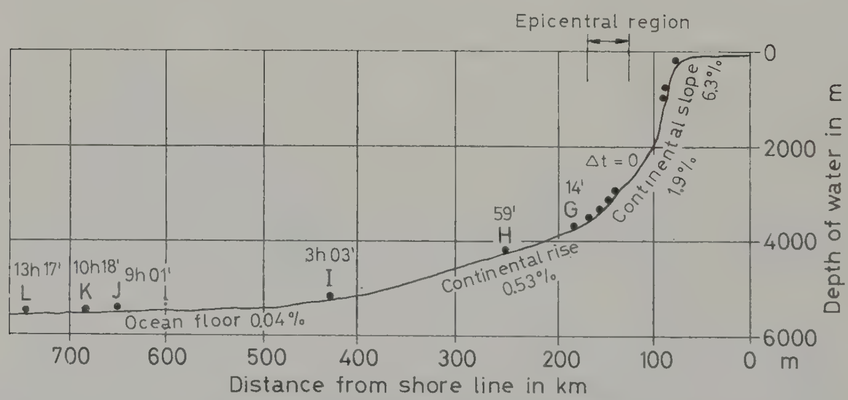


Fig. 5–71. Section through the slope on Grand Banks south of Newfoundland; G to L — submarine cables. The times of cable interruption in relation to the respective distances of the cables from the shoreline give an indication of the velocity of the submarine slide (after Terzaghi, 1957).

of 100 km per hour at the foot of the continental slope. The speed had decreased to 20 km per hour at the end of the current after a distance of 550 km had been covered. Terzaghi (1957) thought that the disturbance had been caused by the liquefaction of sediments beneath the ocean floor in a way similar to the event that broke cables in the Orkdals fjord in Norway, 19 km south-west of Trondheim. However, an examination of six cores taken from the sea floor near the southernmost cable suggested that the former interpretation was the more plausible (Heezen et al. 1962).

One of the largest submarine slides (which was identified by reflection profiling and surveyed with multiple crossings) is the Bassein slide in the Bay of Bengal off the coast of Burma (Moore 1978). The slipped mass covers an area of 3,940 km^2 and has a volume of some 960 km^3 – 80 times larger than the largest Alpine landslide at Flims which has a volume of 12 km^3. The slide occurred on an accretionary slope over the Sunda arc subduction zone. Two factors are thought to have been responsible for the origin of this huge slide, viz. a strong earthquake and a rapid loading of the tectonically disturbed slope sediments which were deposited by the river Bassein, a distributary of the Irrawadi, on the ocean floor in glacial times.

Large submarine slides are also known in the Gulf of Mexico. They were first recognized from the topography and later established with greater precision using high-resolution reflection profiling (Walker and Massingill 1970). Particularly favourable conditions for submarine slumping were present during the Pleistocene at a low eustatic sea level.

A very young submarine slide in the Kayak Trough in the Gulf of Alaska was probably triggered by the earthquake of 1964; the slide is 18 km long and 15 km wide and has a volume of 5.9 km^3. Surprisingly, it moved down a slope with a gradient only 1°. The sediment involved is clayey silt of low strength. As similar sediments also occur in other parts of the Gulf of Alaska, serious problems may be encountered there in laying out oil pipe lines or siting platforms for oil exploitation. The high frequency of earthquake events makes the situation even more difficult (Molnia et al. 1977).

Of considerable interest is the relationship between plate tectonics (or more strictly the tectonics of plate margins) and the frequency and extent of submarine slides but this topic is beyond the scope of our book; we refer the reader to the paper of Moore (1978), who discusses the subject in great detail.

Subaqueous slides have been identified in almost all geological formations, especially in the flysch sedimentary facies (Książkiewicz 1958b). Röhlich (1963) has recorded the slide structures in the area of the Postspilitic Group of Algonkian age in the middle Vltava valley, and Petránek (1963) has studied those in the area formed of Ordovician shales and quartzites.

Several interesting examples of subaqueous slides were mentioned by Hadding (1931) in the Rhaetic claystones and sandstones found in the Scania region of Sweden. Brown (1938) described traces of numerous subaqueous slides involving Tertiary claystones and sandstones, discovered in the oil wells of Ecuador.

Chapter 6

METHODS OF LANDSLIDE INVESTIGATION

The various types of slope movements have been reviewed in the preceding chapter. The engineering geologist faced with any landslide problem has to evaluate exactly a potential or even initiating failure and decide what stabilization measures to apply. To this end, methods for investigating the geological, hydrogeological and mechanical aspects of the problem are necessary. In gathering the requisite information, current investigation methods as well as more sophisticated recently developed techniques may be used. The principles and procedures of engineering-geological investigation will be the subject of this chapter.

The analysis and solution of slope-stability problems need to be approached in the right way. For example, the investigation should be based on a working hypothesis which, as the authors hope, might be constructed on the basis of the standard cases analysed in the foregoing chapters. The investigation programme has to take into account not only the geological and mechanical characteristics of a particular slope but also the method selected for finding a static solution. As will be seen in Chapter 7, a choice has to be made between a basically simple method and a more refined method, depending on the working hypothesis of the behaviour of the slope. Since, for example, deformation characteristics need not be examined in the simple analytical methods whereas they have to be known before the complicated methods can be used, both approaches will be assumed in the following review of investigation techniques.

6.1 Field investigation

Any reasonable scheme of the corrective and preventive measures in a landslide or an area susceptible to sliding must be based on a detailed, integrated geological and geotechnical investigation. It is necessary to study the geological structure of the area, the petrographical and physical properties of the rocks, and the local hydrogeological conditions. As the form of a slope is the end product of past geological processes, the morphological history of the slope must also be understood. The investigation is usually conducted in several stages, in the same way as geological investigations for other building projects (Záruba and Mencl 1976).

The first stage is a preliminary investigation which includes the reconnaissance of the terrain in order to establish the extent of the area to be studied, and the drawing up of a program for detailed investigation, decision being made on the distribution of borings, test-pits or galleries, on the application of geophysical measurements if need be, and other exploratory techniques.

In the second stage the area should be surveyed if suitable topographic maps are not available; it must be geologically mapped in detail, and/or mapped with the use of aerial photography. Test borings, the sampling of rocks and geophysical measurements are carried out at the same time. The second stage is concluded with a summary report on the investigation results, pointing out the main principles of the correction or controlling works.

The third stage involves geological co-operation in the corrective works, the surveying of temporary outcrops, the checking of the conclusions of detailed investigations and, if need be, decisions on supplementary research work that is deemed necessary.

In the fourth stage the long-term effectiveness of the corrective works is monitored and a watch is kept for signs of change in the hydrological conditions, and/or ground movements. The purpose of this surveillance is to ensure that maintenance of corrective installations is carried out in good time.

6.1.1 Survey of the landslide area

By a thorough preliminary examination of the slope, the extent of the hazardous area is determined, and a contour map usually on a scale of at least 1 : 5,000 or larger is constructed by means of current surveying methods. A perfect topographical base for landslide mapping is provided by plans obtained by the photogrammetric technique as these show all the relevant details of the ground surface such as fractures, scarps, and changes in the vegetation. This is of great assistance in the geological investigation of a landslide.

If a tachymetric method is used, a true picture of the relief of the slide area requires a dense network of measurement points and a detailed sketch of the terrain.

Should a reconnaissance survey be all that is required, a detailed survey of the whole area and the compiling of a contour map having to be ruled out for lack of time, then at least several sections must be surveyed. The sections must be long enough to extend into the undisturbed areas above and below the slide. The surface of the area should not be shown in an oversimplified form, but should be represented with as many topographical features as possible, such as all marked edges, swells and depressions, scarps and cracks. The information obtained from surveyed sections is supplemented by the logs of boring.

6.1.2 The use of aerial photographs

The methods of aerial photography and the detailed geological interpretation of aerial photographs have been described in detail in many textbooks, to which reference is made for further information.

The advantages of air photography in the investigation of landslides arise from its economy of time and money, since it covers a relatively large area in one picture and is the only technique that provides a three-dimensional view of the terrain. Using appropriate photo-interpretation procedures an experienced geologist can define precisely the boundaries of landslides; the scarp of a recent slide appears sharply outlined against the adjacent intact area, the slope below the scarp is irregularly undulated with water-filled depressions here and there, and the characteristic cracks are clearly discernible. The amount of movement is easily determined from the offset of linear features such as roads, highways, railways, tracks, etc., as long as they continue into undisturbed areas (Fig. 6–1).

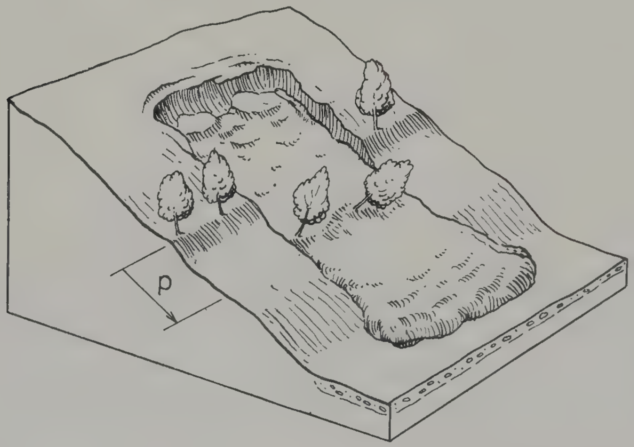


Fig. 6-1. The amount of movement (p) can be determined from the displacement of linear features.

For a geological investigation of a slide or slide-prone area, vertical and oblique black-and-white and colour aerial photographs are taken. Until recently black-and-white photographs were most commonly used. They are taken so that when observed through a stereoscope, they provide an exaggerated spatial model of the area giving a clear picture of the geomorphology. Geological features are deciphered using diagnostic characteristics such as the photographic tone (i. e. the different hues of grey), the structure of the picture, the shape of the features portrayed, and their dimensions and interrelationships.

At present, the use of natural colour photographs is preferred. One of the diagnostic characteristics is then the colour of the objects in the picture. The colour photos are particularly valuable for assessing differences in moisture and drainage conditions, and for identifying soil or rock material and the type of vegetation cover. Multispectral false-colour photographs are taken on films with three emulsion layers sensitive to green, red and infrared rays, but the positive does not show the area in natural colour.

The results of aerial mapping certainly form a very useful basis for more detailed investigation, but it should be emphasized that they are not a substitute for field studies, even if the photos are as perfect as possible and the interpretation is most conscientiously carried out.

Aerial photography is of great assistance in preparing the program of surface exploration, especially with respect to the surveying plan, the lay-out of cross-sections and the selection of positions for boreholes and test-pits. If older photographs of the landslide are available, the progress of the slope movement may be followed by comparing them with new pictures. Photographs with a scale between 1 : 5,000 and 1 : 10,000 have proved the most suitable for the study of landslides.

More sophisticated remote-sensing methods include satellite imagery, infrared imagery and microwave radiometry, the last of which still needs further development and is still too costly for landslide study. Satellite imagery has been found to be convenient for direct recognition of extremely large slides only. Very good results are obtained by infrared imagery, particularly when combined with aerial photography; this method provides information on the surface and near-surface moisture and drainage conditions, on the presence of bedrock at or near the ground surface, on the presence or absence of a loose material cover and of rocks susceptible to sliding, and on changes in soil surface temperature, which seem to be related to those of the water content of the soil (Rib and Ta Liang 1978).

6.1.3 The use of geological maps

For a general information about the potential for landslide occurrence geological maps may be used, which are now available in most countries. They are usually large-scale maps showing only solid geology without the superficial deposits. From these maps we can see that areas of low relief and formed of the solid rocks of old geological formations (as for example the Bohemian Massif) are of low landslide incidence. In contrast, the probability of mass movements is high in young sedimentary (e. g. Cretaceous and Tertiary) rocks, particularly where the relief is deeply dissected.

The most suitable map for the preliminary study of a landslide is a modern engineering-geological map but such maps are so far available only for some areas. They depict both the bedrock and the superficial deposits, and landslides are shown by special symbols. Since, however, the slope movements develop and change with time, the older maps may only be relied upon for general information.

In several countries where slope movements present a serious economic problem, a systematic mapping and registration of landslides has been carried out in order to prevent the building up of hazardous slopes. In Czechoslovakia all important landslides were registered between 1962 and 1963 (Rybář et al. 1964, Záruba and Mencl, 1969, Pašek 1975). Similar investigations were undertaken in Italy in Calabria (Carrara and Merenda 1976), and in the U.S.A. in California (Nilsen and Brabb 1973).

In Czechoslovakia all the landslides studied have been plotted on maps of scale 1 : 25,000. The larger landslides are plotted according to their actual dimensions and the smaller landslides are indicated by special symbols. Each landslide on the map is assigned a number and characteristic data are entered on the registration forms.

By means of different symbols, active, dormant (temporarily inactive), and stabilized (fossil) landslides are differentiated on the maps. Slopes susceptible to sliding are also plotted. Fig. 6–2 shows the symbols used to represent the landslides on a 1 : 25,000 scale map. Active landslides and landslide-prone slopes are depicted in red and structures endangered by slope movement are countoured in red.

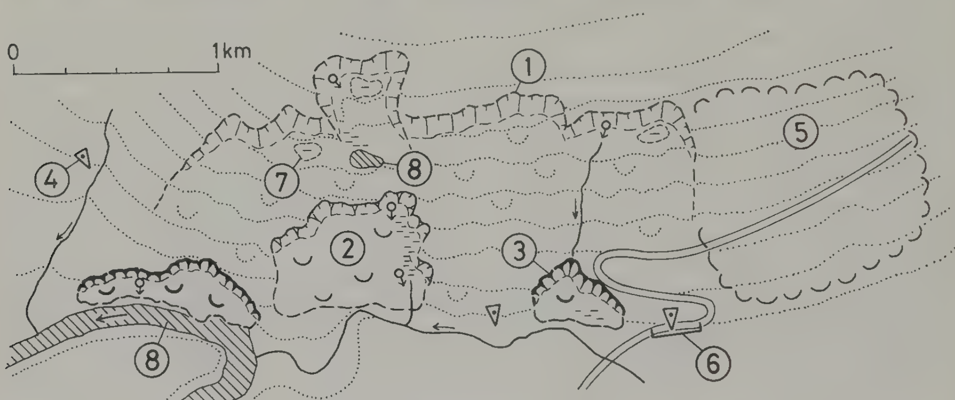


Fig. 6–2. Representation of various landslide types on a 1 : 25,000 scale map; 1 — head scarp of a fossil landslide, 2 — dormant landslide, 3 — active landslide (in red), 4 — symbol for small landslide that cannot be drawn to scale, 5 — landslide-prone area (in red), 6 — structures threatened by slope movement, 7 — depressions, 8 — lakes and streams (Rybář et al. 1965).

The map also contains detailed hydrogeological information (the locations of springs, wet grounds, drainless depressions) which is decisive in the assessment of slope stability since water is usually the main factor responsible for slope movement.

The system of landslide mapping used in Calabria (Carrara and Merenda 1976) is much more complicated; it indicates not only the main types of slope movement but also the ages and depths of the landslides, so that 42 symbols are used, which can be further combined. The compilation of such a map demands the widest experience of the field geologist. A great wealth of information is contained on a single sheet but it is rather difficult to read.

Maps of geologically hazardous zones. In an endeavour to provide as speedily as possible information required for the economic planning of new settlements in mountain regions, the French Geological Service prepared maps showing those zones where geological hazards are to be expected (Cartes de risques géologiques).

Slopes threatened by rockfalls, landslides, erosion, debris flows and earthflows are plotted on a topographic map with a scale of 1 : 20,000. Hazardous places such as head scarps are shown in red shading and with the use of symbols, and green-coloured areas represent places of possible accumulation of moving masses.

The maps are compiled from the results of field mapping and from the study of air photos without any new subsurface exploration. Thus in comparison with engineering geological maps these maps can be prepared at much less cost and within a much shorter time. Their quality of course depends on the experience and expertise of the mapping geologist, and they can never fully replace engineering-geological maps. They are however a suitable aid in maintaining control of geologically hazardous areas, particularly in the mountainous regions (Goguel and Humbert 1972, Humbert 1977).

6.1.4 Directions for the detailed geological mapping of landslides

Although landslides occur in a great variety of forms, as we have seen in Chapter 5, three definite zones can be distinguished in most landslides:

1. the break-away zone (head area) represented by a scarp or depression from which material has moved down;
2. the zone of transportation, in which the principal movement has taken place;
3. the zone of accumulation, where the displaced material has accumulated.

These three zones exhibit different features which must be plotted on the map, viz. the shape of the head scarp, the outline of the zone of accumulation, the course of fissures, the shapes and sizes of individual blocks, etc. In addition, the sites of test-pits, borings and wells and the state of the vegetation (e. g. leaning trees) have to be depicted. In particular, the depth and the dip of sliding surfaces must be measured where they are exposed. The head scarp is represented by the outcrop of the main slip surface along which the rock mass has moved down. If the landslide is not old enough for the initial form of the scarp to have been obliterated by the disruption of the upper edge, then the inclination of the sliding plane can be inferred from the shape of the scarp.

An important characteristic of a landslide slope is its shape in cross-section. If the slope was sculptured by erosion and covered with waste redeposited by rainwash, the profile line forms a gentle curve where it merges with the floodplain. Even a very ancient landslide is recognizable from the convex shape of the toe of the slope. On the basis of this particular shape, an accumulated slipped mass will be readily distinguishable from, for example, an alluvial cone.

The examination of landslide cracks is of special importance for identifying the type of movement involved. The head area is delimited by open cracks (lunar cracks) approximately perpendicular to the direction of movement (Fig. 5–26). In the first stage of movement the positions of the cracks indicate the outline of the head area;

their course in older landslides gives clues to the advance of the landslide headward. Discontinuous cracks arranged "en echelon" and appearing lower down the slope, indicate the lateral extent of the landslide even if the movement is still hardly perceptible.

Fig. 6–3. Slickensided lateral ridges of the Riečnice earthflow in the Orava area of Slovakia (photograph by Rybář).

On the landslide body itself, a series of transverse cracks is observable which in the upper part are generally open and of the tension type. In the lower parts the cracks are closed and sometimes even deformed by pressure. The accumulation zone of a slide is occasionally cut by cracks arranged radially in relation to the arcuate outline of the mass. On both flanks of a landslide longitudinal shear cracks develop along which lateral ridges may be squeezed out. After movement has subsided, striation parallel to the direction of movement is observable on the ridges (Fig. 6–3).

Rock material can move downslope as large discrete blocks, as an agglomerate of small lumps or as a plastic, semifluid mass. Accordingly, the surface of the landslide can be continuous (as in the case of mudflows), hummocky, stepped or terraced.

For effective corrective treatment it is necessary to determine whether the landslide in question is *active*, temporarily *inactive* (dormant), or *stabilized.* For the purposes of mapping, landslides are divided according to their age and stage of development. Age is determined on the basis of geomorphological features, the relationship be-

Fig. 6–4. The stretched roots of trees in fissures are indicative of an active landslide (photograph by Záruba).

tween the slide body and the superficial formations of known age, and the state of the vegetation cover.

Active landslides are characterized by a fresh appearance; the head scarp is steep and free of any vegetation cover, cracks are open, tree roots are strung (Fig. 6–4),

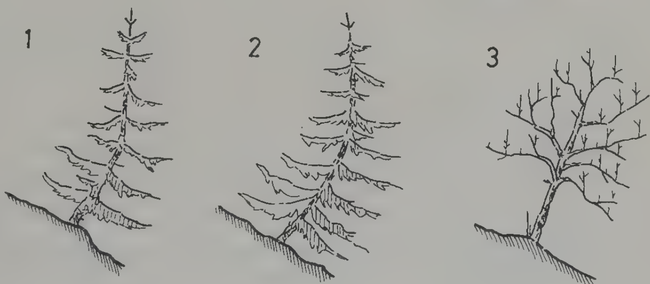


Fig. 6–5. Disturbance of vegetation cover reveals recent movement; the time of the movement is obtainable from the curvature of trunks and new growths.

roads etc. are interrupted and buildings more or less damaged. The state of tree growth is indicative of the age of recent sliding movements. Trees which have tilted down the slope on unstable ground (Fig. 6–5), continue to grow vertically during the period of inactivity so that the trunks become conspicuously bent. From the younger, vertically growing part of the trunk, the date of the last sliding movement can be deduced.

The outlines of dormant landslides become partly obscured, the scarp becomes extensively overgrown and after the tongue has suffered from erosion it becomes indistinct or even buried by alluvial cones (Fig. 6–6).

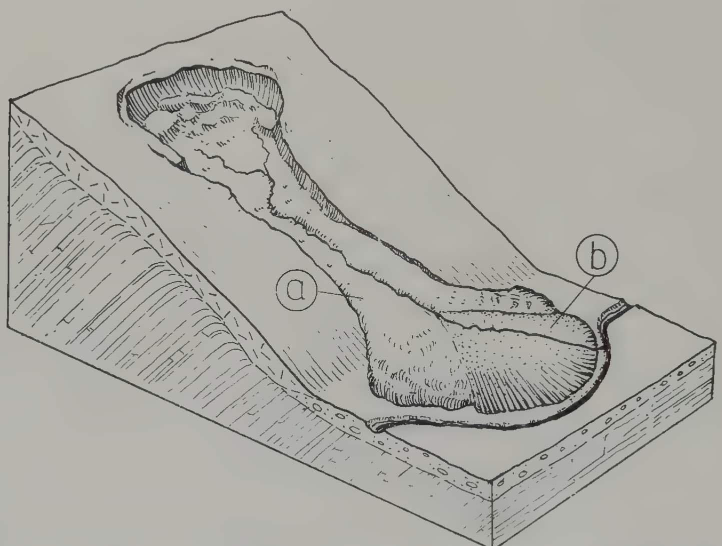


Fig. 6–6. Ancient earthflow with the tongue (a) partly buried by a younger alluvial fan (b).

The age of a landslide can also be established from its relationship to the river terraces or loess sheets. Terrace aggradations, when disturbed by slipping, occur at levels differing from the positions which would be expected from the long profile of the valley. A terrace affected by slide movements provides evidence that the movements postdate its formation; if the unevennesses generated by this disturbance are levelled out by loess loams, it is evident that the movement occurred earlier than the deposition of loess.

Illustrative examples of these interrelationships may be taken from the Turnov area in north-eastern Bohemia. Pleistocene gravels lying on Cretaceous marls have been affected by slope movements in the Libuňka river valley. Steps formed by the movements were levelled out by younger loess deposits. The slope movements occurred in the younger Pleistocene, after aggradation of the terrace but before deposition of the youngest loess (Fig. 6–7).

Loess generally forms drifts on the lee-sides of head depressions and landslide tongues. In this way the Pleistocene age of some landslides has been ascertained (Fig. 6–8).

The recent development of a soil profile provides another clue to the age of a landslide. If in the head area an undisturbed soil profile (e. g. a brownearth) exists above a loess cover, the landslide can be regarded as practically at rest because a soil profile needs a period of several hundred years for its development. On the other hand,

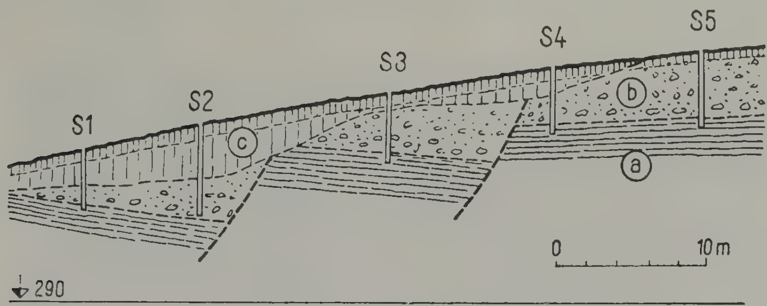


Fig. 6–7. The terrace gravels (b) lying on Cretaceous marls (a) have been affected by slope movements. The Pleistocene age of the movements is evidenced by the intact younger loess cover (c).

a step-like, broken, or otherwise affected soil profile points to relatively recent movement. Fossil soil profiles can occasionally serve as a means of dating slide movements, too. Thus, for example, in the valley of the Chomutov brook in the Žatec area of

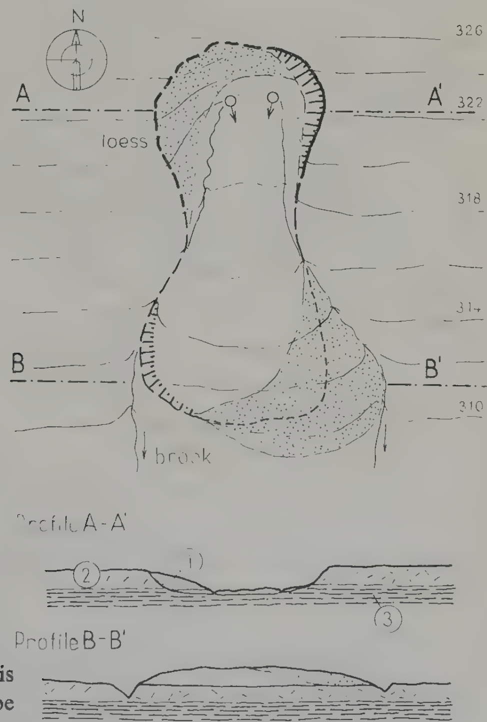


Fig. 6–8. The Pleistocene age of landslides is evidenced by loess covers; 1 — loess, 2 — slope debris, 3 — Neogene clay.

western Bohemia, buried soil profiles along with terrace gravels show a step-like disturbance caused by sliding, whereas the recent soil profile (degraded chernozem) which has developed over the youngest loess sheet is continuous, and shows no sign of disturbance. This observation suggests that the movements took place in the youngest Pleistocene, the slope having remained at rest since the deposition of the younger loess layer (Fig. 6–9).

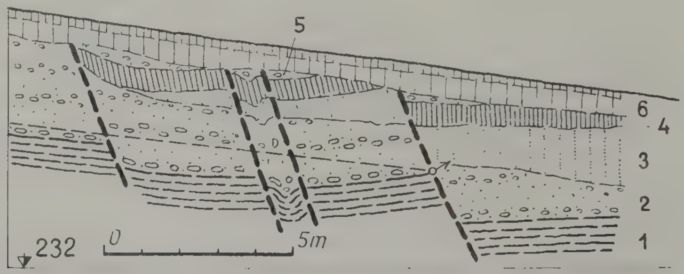


Fig. 6–9. Section through the head area of a fossil landslide exposed in a sand-pit near Žatec in western Bohemia. The undisturbed layer of the youngest loess indicates the Pleistocene age of the movements; 1 — Neogene clays, 2 — Pleistocene sandy gravels, 3 — older loess, 4 — fossil chernozem, 5 — drawn-out sandy gravels, 6 — younger loess.

The presence of characteristic plant species can be of help in recognizing and delimiting areas prone to sliding. Sýkora (1961) has found that potential landslide slopes tend to be overgrown with a particular flora, represented by horsetail (Equisetum) and coltsfoot (Tussilago).

According to our observations, there indeed is a very conspicuous relationship between landslide occurrence, mainly on glauconitic rocks, and horsetail growth (Fig. 6–10). Almost all of the ancient earthflows in Slovakia and many of the landslides in the Cretaceous areas of Bohemia are covered with a dense mat of Equisetum maximum. A growth of Equisetum maximum may also be observed on the sliding slopes in Rügen island in the Baltic sea. Sýkora (1961) attributes the wide distribution of Equisetum maximum on landslides to spreading out of its jointed deep-seated (at the ground-water level) rhizomes over the sliding slope as a consequence of soil movement. In the moist soil the rhizomes regenerate readily and give rise to new plants. In the slide area near Dubková in eastern Moravia, horsetails were originally confined to the top of the affected area but the movement that occurred in 1937 enabled the plants quickly to colonize the entire slope so that it forms now a dense cover and impairs the value of the land as pasture. Being strongly siliceous, the horsetail is avoided by cattle, and this contributes to its ability to spread.

The occurrence of these plants is also caused by the geochemical conditions of the habitat. Chemical analyses have shown that they contain 50–60% SiO_2 and 19–30% K_2O in the ash (Němec et al. 1936). The SiO_2 content varies considerably being as high as 83.5% in the ash of some specimens of Equisetum palustre. Lindstow

(1929) likewise reports a value of 40 – 96% SiO_2 in the ash of horsetails. Thus, horsetail growth is indicative of the presence of potassium and hydrated silicates from which SiO_2 is liberated by alkaline water. For this reason, Equisetum (in particular E. maximum) is a good indicator plant of slide areas in potassium-rich, e. g. glauconite-bearing rocks.

Fig. 6–10. Horsetail growth (Equisetum maximum) in the head area of a landslide near Děčín, northern Bohemia (photograph by Rybář).

The depth of the slide plane as well as the age of slope movements can also be determined by palaeontological methods. The displacement of fossiliferous rocks can be gauged from the mode of preservation of the fossils, from their proportional representation or from changes in their chemical composition (e. g. decalcification). Vitally important clues are given by the presence of fossils which are not indigenous to the rocks in question. Thus, for instance, Quaternary fossils occurring in Cretaceous marls indicate that the marls have been disturbed and displaced by slope movements. The original depth of slipped marlstones near Březno in northern Bohemia (Fig. 5 – 27) became evident after the find of Quaternary gastropods which were displaced along the slip surface to a depth of 12 m.

When taking samples for palaeontological investigation their geological position must be thoroughly established; if the fossils are found, for example, in the secondary filling of a fissure, the results of the investigation might be greatly distorted without careful noting of this fact. The age of displaced rocks can be deduced from their relationships with palaeontologically dated beds. Taking the fossil landslide near Mikšová on the river Váh as an example (Fig. 5–36), the slope movement could be placed in the interval preceding the last interglacial, as the silty alluvium deposited at that time covered the toe of the slide.

In block slides, the slipping of solid rocks on a soft substratum leads to the opening of fissures which are subsequently filled with younger sediments. When the latter are datable by palaeontological or archaeological finds, the upper age limit of the slope movement can thus be established.

Drainless depressions on sliding slopes are often overgrown with fens and peats, which makes it possible to ascertain the age of sliding movements by pollen analysis.

Once the slope has been surveyed and mapped in detail, some practical questions can readily be answered. Thus, the shape of the slide surfaces can be reconstructed, and consequently also the depth of the slide; the number and timing of recurrent movements can be estimated according to the form of the ground surface and the vegetation cover.

6.1.5 Hydrogeological research

Any programme of corrective measures for a landslide requires a good knowledge of the hydrogeological conditions of the slide itself and its wider surroundings. The first task is to determine the depth of the ground-water table and the range of its fluctuation, and to map all inflows of water into the slide area, including springs, seeps, wet grounds, undrained depressions, aquiferous fissures and permeable strata.

The changes in the relief of the slope produced by a slide alter the drainage conditions of surface waters as well as the ground-water regime. Outlets are frequently obsctructed and the water finds another path through the disturbed rocks. The slip surfaces are generally impervious, retaining both surface water and ground-water; wherever they approach the ground surface, new springs and wet grounds appear. In the boring logs all data on ground-water conditions, e. g. the depth and fluctuation of the water table must be recorded. Special attention should be paid to confined (artesian) water which exerts an uplift on overlying beds.

The pore-water pressure in clayey soils affected by sliding has the same influence on ground stability as the uplifting effect of ground-water. However, this pressure cannot be determined simply by observing the water level in borings, because when the water fills the borehole its pressure in the vicinity of the hole changes. It is therefore necessary to install piezometric instruments for pore-water measurement in separate holes at a good distance from any geological boring, just as is done in the case of impermeable cores or the clay bases of earth dams. It is the task of the geologist or expert in soil mechanics to decide whether the measurement of the water level in boreholes is acceptable or whether a more reliable measurement of pore-water pressure is necessary.

6.1.6 Surveying the slide movement

A systematic survey needs to be counducted if the development of a landslide is to be analysed and the effectiveness of corrective measures assessed. For this purpose several methods are used which allow the absolute or relative displacement of observation points to be determined.

Conventional geodetical methods are most commonly applied and are satisfactory where the rate of movement per year is greater than the error of measurement, i. e. 2.5 – 5 mm.

On the surface of the slide area a network of measuring points is laid out and their displacement relative to fixed points located on the adjacent stable area is measured. The monitoring is carried out either at regular intervals or at times depending on the factors controlling the rate of movement, e. g. the spring thaw, rainy periods and earthquake. It is of course necessary to ensure that the fixed points are really on stable ground and that the measuring points are unaffected by any subsidiary movements which would distort the measurement results.

Measurement of a triangulation net includes the measurement of all angles, distances and heights, and their comprehensive evaluation. This is very laborious and time-

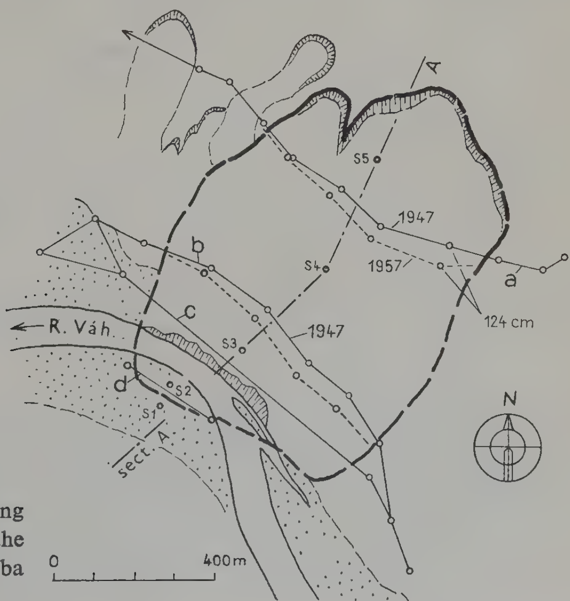


Fig. 6-11. Distribution of measuring chains (a, b) and sight-lines (c, d) on the Sučany landslide (Petrášek and Záruba 1959).

consuming with the result that a succession of measurements is expensive to carry out.

Geodetic surveying is often simplified so that height measurements are made separately and instead of using a triangulation grid the observation points are arranged in geodetic profiles across the slide. In monitoring the movement of the Sučany slide, for example, four geodetic profiles were set up across the slide; horizontal displacements were obtained by simple polygonal surveying and vertical displacements by levelling.

Figure 6–11 shows the location of measurement sight lines and chains on the Sučany landslide. Two former chains (designated by *a* and *b* in Fig. 6–11) were used, these having been laid out during cadastral surveying in 1947, and two new lines, c and d, were laid out. The largest movement was measured at the points in the central section of the landslide. It amounted to 124 cm in ten years. The average rate of movement of 8–13 cm in one year corresponds with the results of surveys during the period 1955 to 1957, i. e. up to the time of stabilization of the landslide (Petrášek and Záruba 1959).

Fig. 6–12 shows the results of levelling along line d which was laid across the rampart pushed forward at the front of the slide. Surveying was repeated four times at intervals of approximately six months. A continuous upward movement amounted

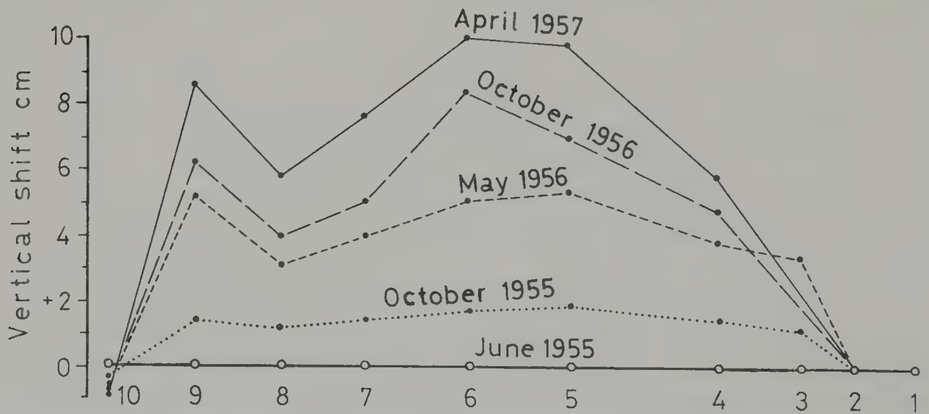


Fig. 6–12. Elevation changes of points on line d on the Sučany landslide from 1955 to 1957 (Petrášek and Záruba 1959).

to·5 cm per year on average, and the horizontal displacement was of a similar value. This finding was consistent with what might be expected from the geological profile, since the slide surface along which the front of the landslide was thrust up, dips at an angle of 40°.

In surveying the rate of movement of the Handlová landslide the observation points were also aligned in three geodetic profiles across the slide body. The measurements carried out between January and May 1961 were repeated several times and the vectors of the displacements are shown in Fig. 5–13.

In the area of Handlová it was possible to compare triangulation measurements made in 1907, 1930 and 1958 with the results of recent surveying; it was assessed that the andesite blocks were moving valleywards at a rate of 15–40 mm per year, and that the movement had continued at a uniform rate for more than 50 years (Malgot et al. 1974). Such long-term measurements are feasible only in exceptional cases, since earlier triangulation grids were not chosen specifically for the purpose of surveying slope movements.

Recently, several advanced techniques have been developed which give more accurate results in a much shorter time. For measuring horizontal distances an electronic distance measurement (EDM) device has lately proved very convenient. It is essentially an instrument which makes possible direct distance measurement by means of electromagnetic waves; laser beams may also be used. The accuracy of the distance measurement methods is to some extent affected by weather conditions.

Novosad (1978) reported very favourably on the Swedish device AGA 700 and 701 geodimeters which he used for monitoring an extensive rockslide on the side of the Šance reservoir in the Flysch Carpathians (Moravia). Figure 6–13 shows the layout

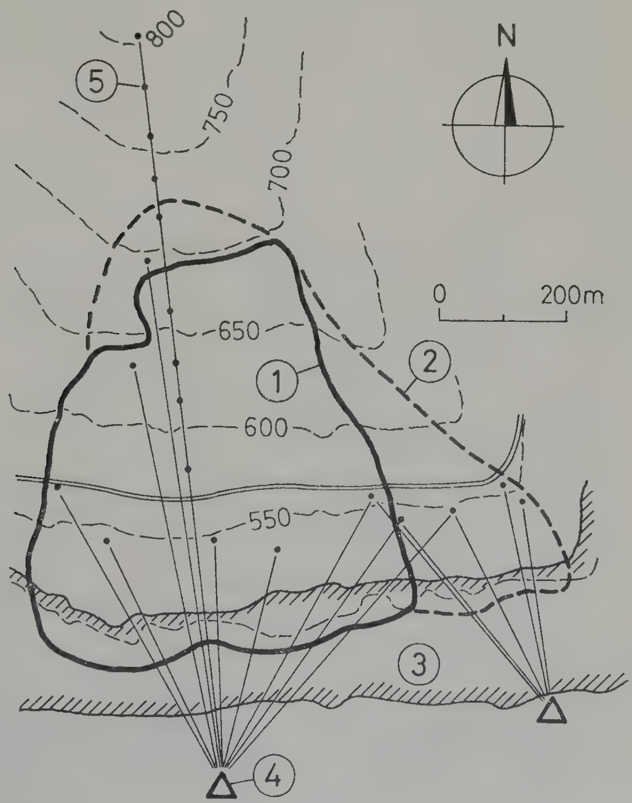


Fig. 6–13. Scheme of the geodetic control of the Rečice landslide; 1 — boundary of the old landslide 2 — demarcation of potential landslide area, 3 — Šance reservoir on the river Ostravice, 4 — benchmarks on the stable slope, 5 — control points (Novosad 1977).

of the geodetic scheme, which permitted the rapid measurement of movement using both the polar method and the method of frontal length intersection. The error in the measurement of the displacement of control points was about 1.5 cm. One great advantage of the technique was that the measurement time was reduced from several weeks to one day.

Another method usable for movement monitoring is the photogrammetric approach. Sequences of photographs taken simultaneously from two or more permanent sites that overlook the slide area allow the displacement to be determined.

A direct measurement of slope movement can also be conducted with the use of simple wire extensometers. In order to establish the displacement, a point on the landslide body is connected by a wire with a benchmark on the stable terrain. Readings are taken mechanically or are registered electrically. The application of wire extensometers is shown in Fig. 6–14.

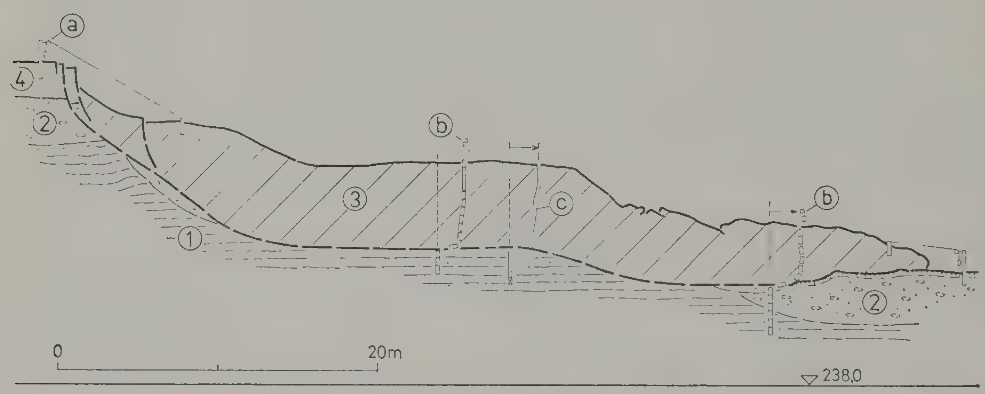


Fig. 6–14. Section of the landslide near Nechranice (after Rybář); 1 — Miocene fissured clays, 2 — sandy gravel, 3 — slipped mass, 4 — man-made ground, a — wire extensometer, b — test borehole with a shear-plane indicator, c — borehole with instrument for measuring pore pressures.

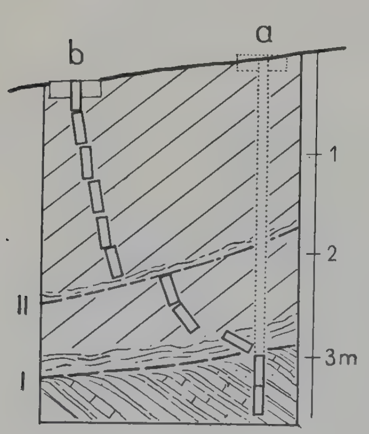


Fig. 6–15. The shifting of bricks in a test pit shows the amount of movement along slip surfaces I and II; a — original position of bricks, b — their position after reopening of the pit.

Observation points are not always set on the ground surface, but are occasionally established at successive depths in a borehole in order to observe the displacement of individual layers. For this purpose wooden or concrete blocks, bricks, or drain tiles are used and the exploratory pit is then backfilled. Upon opening such a pit after a given period, rates of movement at different levels can be inferred from the positions of the respective blocks (Fig. 6–15).

6.1.7 Determination of the depth and shape of a slide surface

The outcrop of the slide surface is invariably indicated by a steep head scarp in the upper part of the slope. The lower end of the slide surface can be exposed above the foot of the slope or in the valley floor. In the latter case part of the floor is heaved up into a rampart in front of the slipped mass.

The orientation and shape of the slide surface control the depth of the landslide. If the layer of moving material does not exceed 1.5 m in thickness, one may speak of a surface landslide, if it is less than 5 m, the slide is described as shallow; a deep slide ranges from 5 to 20 m, and a very deep slide is one that exceeds 20 m. The depth of a slide is usually measured at right angles to the surface of the slide.

The slipping may occur along one sliding surface or may involve a number of slip planes lying one above the other. In large landslides they constitute moulded and kneaded zones up to several metres thick.

For the determination of the course of the sliding surface and the depth of the slide, test pits and test trenches are generally dug since these permit direct inspection of the individual rock strata and easy taking of undisturbed samples for laboratory research. As a rule, testpits and trenches are excavated in slides which are already at rest.

The digging of deep-test pits in loosened rocks which are in motion is very difficult and dangerous. However, the trenches have the advantage that they can be used as stabilization ribs for the drainage and stabilization of a landslide. The trench is generally dug from the lower end in sections 8–10 m long, which immediately after geological appraisal are filled with stone rip-rap or gravel with a sand filter. A continuous outflow of water should be achieved so that the lower part of the trench does not get clogged with sediments precipitated from turbid water. When the trench remains open over a greater length, the sides being supported only by timbering, it occasionally caves-in upon reactivation of the movement. For this reason, galleries are sometimes more convenient, and may likewise serve as a means of subsurface drainage of a slide and contribute to its stabilization.

In most cases only boreholes are available for this purpose. These, however, must be drilled with great thoroughness if they are to yield satisfactory results. In a dormant landslide it is very difficult to determine the position of the surface or zone of plastic deformation by boring or with hand-augered holes. In the case of core boring these usually soft zones are disturbed by the drilling fluid, and therefore a double-tube core barrel must be used. It is necessary to study not only the lithological changes in the rocks but also their consistency and water content. Weakened zones along slide surfaces are characterized by a higher water content, some degree of kneading and a lower unit weight of the dry substance. Such zones may only be several centimetres thick and consequently undisturbed samples are not easy to take. If the hole is bored, for instance, by helical auger, the soft kneaded bed is found with comparative ease but it may be bored through with one turn. In this case it is best to take at least disturbed samples of soil from the auger and, by submerging them in hot paraffin or fluid rubber, prevent any loss of moisture and consistency. If undisturbed samples should be taken from these beds, two holes close together must be bored. The first is used for a precise determination of the position of the kneaded zone, the second for taking undisturbed samples. During this operation ground-water horizons must be carefully sealed-off so that the rocks at the bottom of the boring are not lubricated by ground-water flowing into the hole. Exploration boreholes often penetrate several slide surfaces in succession. For the computation of slope stability and selection of appropriate remedial measures it is necessary to be certain of the depth of the deepest

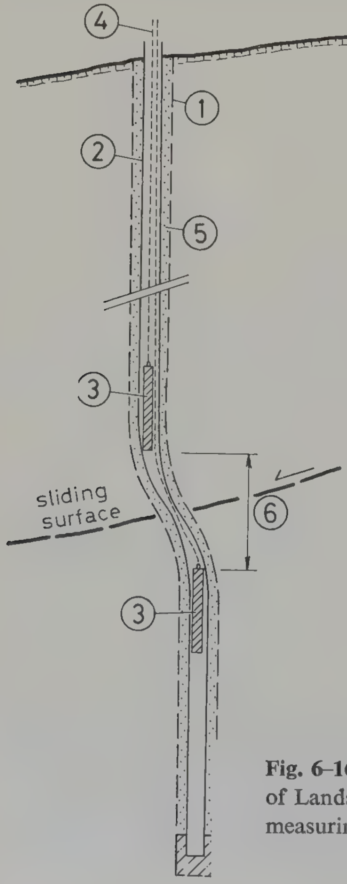


Fig. 6–16. Measurement of the depth of the slide surface (Japan Soc. of Landslides, 1972); 1 — borehole, 2 — plastic protecting tube, 3 — measuring mandrel, 4 — wires, 5 — sandy backfill, 6 — bending of the tube.

slip surface. The slip surface which coincides with a bedding plane or other plane of separation is not difficult to determine. In a landslide in which slip surface is irregular, boring in both longitudinal and transverse directions must be carried out.

The depth of the slide surface in an active landslide can be determined more readily, because the hole develops a deflection at the level of this surface. Deflections arising in bores have long provided a means of locating slide planes as bars, plumbs or rods when lowered down became stuck at the point of inflection.

The demand for greater accuracy has led to the development of several new methods. As a rule, separate holes are drilled and are equipped with appropriate measuring devices. An active slide surface can easily be located with the use of two lead mandrels. A thin-walled plastic tube (about 1.5 cm across) is set on the bottom of the borehole and sand is packed around it. One mandrel is lowered on a wire into the tube and the other mandrel is likewise passed down for each measurement (Fig. 6–16). When the movement is renewed, the tube bends at the slip surface and the mandrels cannot pass through it. If the movement occurs at more than one surface, only the positions of the uppermost and deepest slide surfaces can be located (Japan Soc. of Landslides 1972). A disadvantage of this device is that it does not give any indication of the direction of the slide movement. For this purpose inclinometers of several types have been developed.

For inclinometer measurements holes of a larger diameter (up to 10 cm) are bored so that an inclinometer of the type used for measuring the curvature of boring can be inserted. The Slope Indicator Co. of Seattle in the U.S.A. produces aluminium tubes with four inner grooves to ensure that the inclinometer is oriented precisely inside the tube. It is thus possible to register any deflection from the vertical in two perpendicular directions. The device makes it possible to take measurements over the entire length of the tube and to draw a continuous profile of the deflected hole, unless the tube is sheared off by the slide.

It is also possible to determine whether the movement occurred along several successive slide surfaces or whether it was of a viscous flow character without developing any marked slide surface. Knowing whether a simple surface or a thick kneaded zone is involved is particularly important with deep landslides, because in the latter case the strength and other physical properties of soils may differ widely from values established by current laboratory tests.

Results of high accuracy are obtainable with an inclinometer which is manufactured by Interfels and referred to as a "*digital inclinometer*"; this instrument makes it possible to monitor movements that are perpendicular or oblique to the borehole axis. The device has a rigid basal section 1 m long, and during operation it is secured at the ends to the borehole walls with pneumatic centralizers (Müller et al. 1977). For making determinations in the case of an active slide surface Interfels also supplies the "multipoint deflectometer". This device combines the principle of the inclinometer with that of the extensometer. It can be installed in a borehole of any inclination and consists of a system of measuring cells connected by rods or wires. Any changes in the original angles of coupling between pairs of cells are converted into electrical signals. Simultaneously, changes in the distances between pairs of cells are also registered. The instrument is suitable for the measurement of small deformations only, and its use is limited by its high cost.

For the determination of the depth of a slide surface and the rate of movement a "*shear-plane indicator*" has proved suitable. This device of which the original version was developed in Czechoslovakia (Rybář 1968) consists of several wires positioned at different depths in the borehole, particularly where the active slide surface is expected to be (Fig. 6–17). If movement occurs at a surface penetrated by the borehole, all wires below the slide surface will show a double bending, whilst those above it will remain straight. As a result of the bending, the upper free ends of

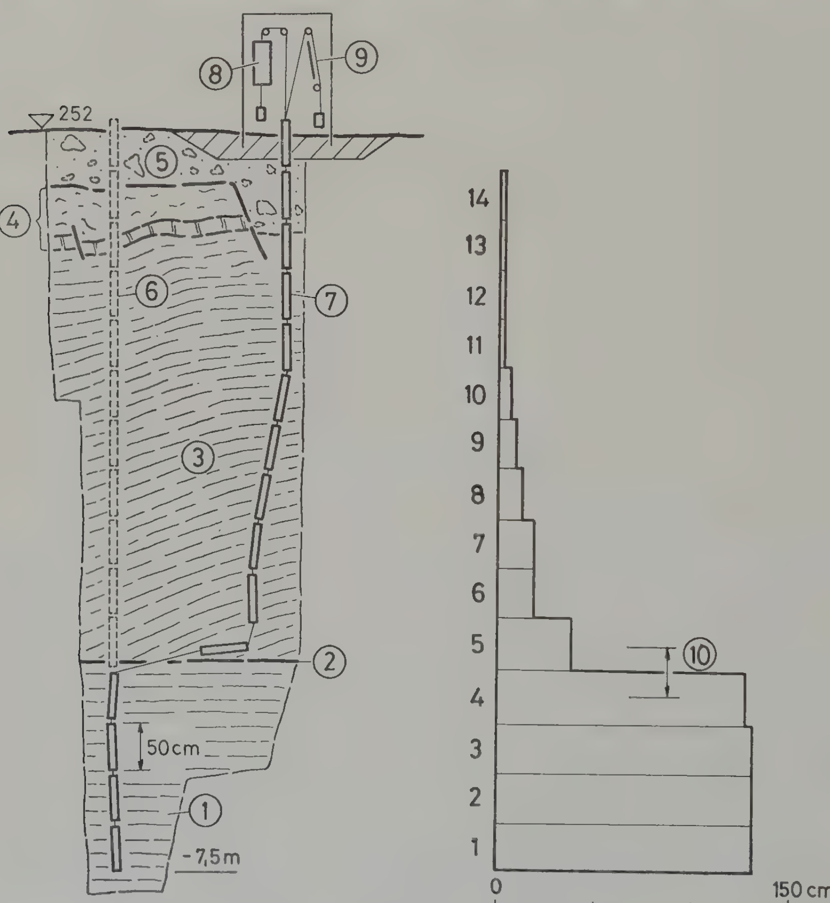


Fig. 6–17. Use of a shear-plane indicator and graphic representation of measurement results (after Rybář), 1 — Miocene fissured clay, 2 — shear plane, 3 — slipped clay, 4 — weathered clay, 5 — sandy gravel, 6 — original position of tubes, 7 — position of tubes after movement, 8, 9 — registration equipment, 10 — location of the shear plane.

wires that are anchored below the shear plane are pulled into the borehole. The extension of the wires is easily measured at the ground surface (Fig. 6–18). The drawback of this technique is that the instrument begins to record the movement only after the displacement has exceeded the inner diameter of the guide tubes or borehole profile. The device may be used wherever movement occurs along a discrete slip

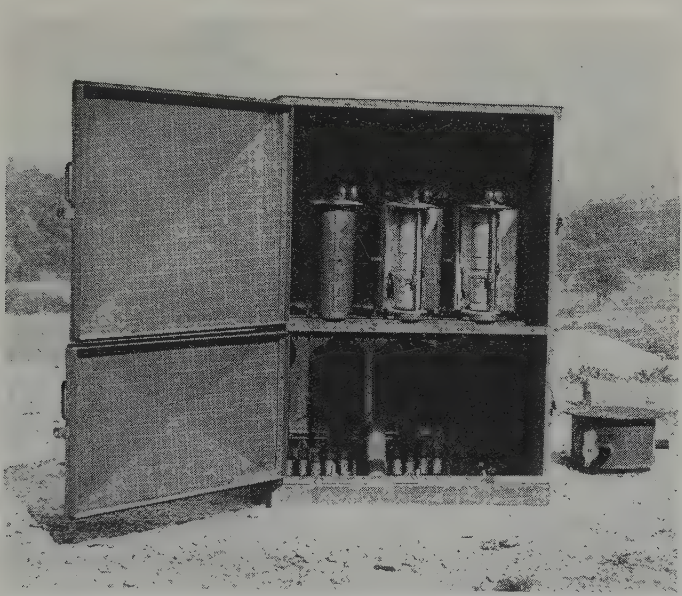


Fig. 6–18. Device for the measurement of displacement in an active landslide by the downward pull of wires fastened below the slip surface (from Rybář).

surface or a thin shear zone; its performance is poor when used for monitoring a mass movement of the creep type. The shear-plane indicator was used first by Rybář (1968) for tracing the slide surfaces in the Nechranice landslide. Kirschke (1977) also used it for the investigation of a landslide in Greece, where the shear plane was at a depth of some 100 m.

Another method suitable for determining the onset of the sliding movement and the depth of the slide surface is based on the breaking of circuits between strip conductors. The device constructed at the Institute of Geological Engineering in Brno (Czechoslovakia) consists of insulated wires joined to form a strip which is placed in a plastic tube. The strip is doubled back at the bottom of the hole so that both ends are at the ground surface. The plastic tube containing the strip conductor is filled with a cement grout. Upon hardening of the grout the strip conductor and the tube solidify into a compact mass, which is displaced as an intact unit when a slope movement occurs. At various depths connecting bridges between pairs of adjacent conductors are located. The connecting bridge situated nearest to the shear plane can be determined by testing for loop continuity at the conductor ends. The device also can be made to record the moment of conductor severance, i. e. the moment of inception of movement, as well as the depth of the slide surface. The apparatus has the advantage of being cheap and easy to install and handle.

For the measurement of slow creep movements Ter-Stepanian used wells or large-diameter boreholes lined with concrete rings; the centre of each ring was marked by crossed threads. Any shifting of the upper ring relative to the lower one is indicated by a pendulum. Unless the well is naturally dry, the water must be pumped out before each measurement.

Relative displacements in cracks and fissures can be revealed by the use of dilatometers. There are many versions of this instrument based on mechanical, electrical or other principles. A dilatometer for the measurement of very slow movements such as the opening of joints between sliding blocks, has been developed in the Geological Institute of the Czechoslovak Academy of Sciences. It has been constructed according to an original principle and makes possible three-dimensional measurements (Košťák

Fig. 6–19. A target-meter TM 71 (Košťák 1969) and a dial-gauge rod dilatometer Holle (G.D.R.) used for measurement of relative movement between two blocks at Tupadly in Bohemia (photograph by Rybář).

1969, 1977). The device was designed on the Moiré principle in which movement is assessed from the fringe patterns produced by grids displaced relative to one another. The extensometer consists of two rigid holders and two Moiré units, i. e. two pairs of metal plates with glass spiral grids. The holders are fixed to the facing walls of the joints between the blocks; the units are mounted on the edge plates of the holders, the upper unit in a horizontal position and the lower one in a vertical position. The displacement is thus detected in two perpendicular planes, and it is evaluated from the total number of fringes. Readings can be made either visually or photographically. The sensitivity and accuracy of the instrument depend on the density of the grids, an accuracy of 0.03 mm having been achieved so far (Fig. 6–19). Very slow slope movements of the order of 0.1 mm/year have been detected despite temperature interference (Košťák and Avramova 1977).

6.1.8 Field investigation of the mechanical properties of rocks

Owing to the great variety of landslide phenomena (discussed in chapter 5) and the miscellany of factors responsible for diminishing the stability of slopes, static solutions to slope stability and the improvement of slope integrity are often unreliable in complex geological situations and the importance of static solutions has been pushed rather into the background. In the case of rock slopes, doubts have arisen as to whether static solutions can serve as a basis for analysis at all, considering the variety of unknown parameters and their interrelationships (Bjerrum and Jörstad, 1966). Nevertheless, static solutions are often of great assistance and in many cases they may give an accurate picture of slope stability. They do at least draw attention to several factors which might escape the attention of the less highly trained investigator.

For the purpose of static analysis the mechanical as well as the index properties of rocks and soils must be investigated. It is not the aim of this section and sections 6.3.4 and 6.3.5 to analyse all those properties which are usually investigated. Field and laboratory works are often carried out as a matter of course, and several properties are investigated without considering the geotechnical aspects of the problem. We prefer to draw attention to the need for a more direct approach based on a purposeful investigation.

Several of the important properties are preferably determined by in-situ tests which will be the main subject of this chapter.

6.1.8.1 Deformation properties

The importance of deformation properties increases when the more elaborate methods of slope analysis are applied. However, differences in the values of these properties are less important in the case of hard rocks and therefore tests on weak rocks take priority. While the deformation properties of soils can be determined by laboratory techniques, field tests are important because samples of laboratory dimensions are not representative of large masses of weak rocks.

Field plate loading tests are widely used, but in the case of weak rocks, correct interpretation is not easy to achieve. The results of an analysis of tests on siltstone carried out by Mejzlík and Mencl (published in part, 1975) are plotted in Fig. 6–20.

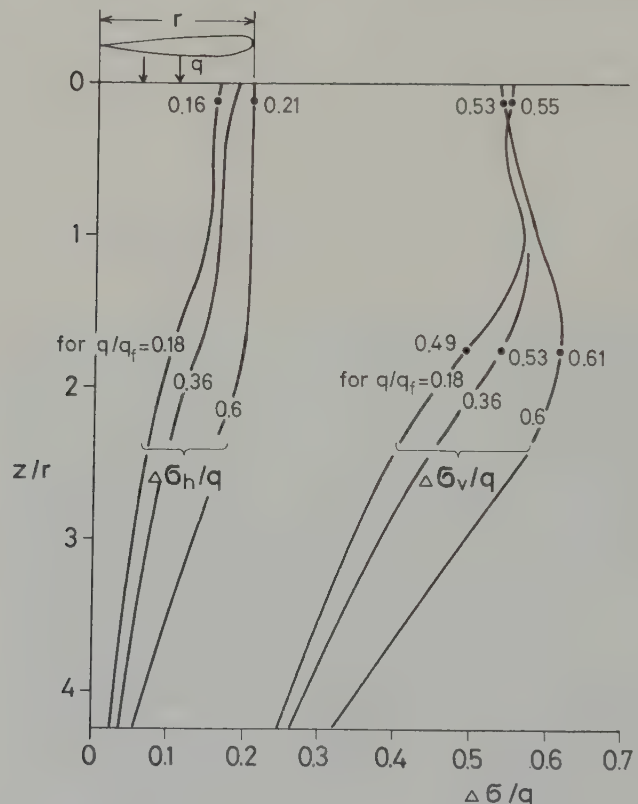


Fig. 6–20. Vertical ($\Delta\sigma_v$) and horizontal ($\Delta\sigma_h$) normal stresses in siltstone under the centre of a rigid plate, produced by uniform load q; q_f — ultimate bearing capacity of the soil (1.35 MPa).

They indicate the magnitudes of the vertical and horizontal normal stresses generated by the loading (q) of a rigid circular plate (radius r) at different depths (z) below the plate axis, for three magnitudes of ratio q/q_f, the latter being the bearing capacity of the rock mass under the plate; its magnitude was calculated by using the classical formulae for bearing capacity (Fig. 10–16 in Záruba and Mencl, 1976). Although more evidence is necessary some provisional conclusions can be made:

(a) The pseudoelastic interpretation (see section 6.2.4) of the plate loading tests by using Schleicher's formula results in the magnitudes of the deformation moduli by about 30 per cent smaller than the real ones, although reasonable values can be obtained in hard rocks. This is caused by the yielding of rock below the periphery of the plate.

(b) Moreover, the safety factors at depths of practical relevancy are small, i. e. of the order of 1.1 to 1.5. Therefore the magnitudes of the deformation moduli, even when corrected with respect to the previous paragraph, are valid for this range of safety only.

(c) To obtain the magnitudes of the deformation moduli valid for greater factors of safety the vertical displacements should be measured at two depths below the plate axis, as already proposed by Shannon and Wilson (1966). Differences in the vertical displacements between different depths can be calculated from the measurements. The magnitudes of the vertical and horizontal stresses generated by the plate can be determined from the diagram in Fig. 6–20, when plates of diameters 0.8 to 1.2 m are considered. From this the moduli can be calculated using Hooke's law, if, as already said, the pseudo-elastic behaviour of the rock is assumed.

The deficiencies of field plate loading tests bring along the tendency to apply other testing techniques. The results of pressuremeter tests have been promising, since they provide higher magnitudes of the moduli. Seismic tests are assumed to provide unrealistic, high magnitudes of the moduli, because they render rather the elastic components of deformations. But the experience shows that the results are not far from reality when the moduli at greater depths are considered.

Cycles of loading, unloading and reloading should be carried out in the course of any type of loading test, partly because many problems of the slope analysis are connected with unloading, and partly because the designer might prefer the mathematical method of analysis dealing with elastic and plastic deformations separately.

The shear moduli. The different directional shear moduli are important in anisotropic rocks; the strains developing along the planes of anisotropy are usually greater than those across the planes, and involve a more plastic type of behaviour. The results of torsion tests conducted on Ordovician shale, were discussed by Hudek (1979). The material behaved isotropically with torsional loading up to about 40 per cent of the torsion shear strength. Above this threshold the torsional moduli began to decrease, and with a loading of 50 per cent of the torsional strength it was less than 25 per cent of the value obtained with a 20 per cent of loading. With isotropic claystone, the corresponding reduction in the torsional moduli (when the loading increases from 20 to 50 of the torsional strength) is about 50 per cent. This kind of anisotropy cannot be analysed in terms of relationships of anisotropic elasticity. Preference is often given to the simulation of anisotropy by introducing small weaker layers into the mathematical models.

Another phenomenon of anisotropy, the difference in the E-moduli if compressed in two directions (normal and parallel to the planes of anisotropy) can easily be investigated by plate loading tests. It is interesting to observe that it is an arbitrary matter as to which of the two is the greater. Mathematical formulation of this behaviour is possible within the framework of the theory of elasticity and therefore pseudo-elastic solutions are possible.

6.1.8.2 Field shear tests

Field shear tests are often performed because laboratory tests are not possible with many rock types, and also because it is believed that they will provide more realistic values than those obtained in laboratory tests. However, when weak rocks and soils are being examined, field tests often give values of resistance that are lower than the real values. There are probably two reasons for this: the values of the safety factors below the tested blocks decrease already during the introductory loading of normal forces and this is accompanied by a disturbance of the rock structure. The subsequent shear loading gives rise to an increase in the compression of the material, and if clayey material is present, high pore water pressures may develop. The tests are also expensive and therefore there is always a strong tendency to reduce the number carried out.

Some use can be made of information on the shear strength of a rock when this is obtained on the basis of its similarity to other rocks. This approach was mentioned by the authors in a previous book (Záruba and Mencl 1976, Table 3—1). Nevertheless, the importance of both the volume increase, and more particularly the volume decrease, which develop in the course of shear straining is still not widely known.

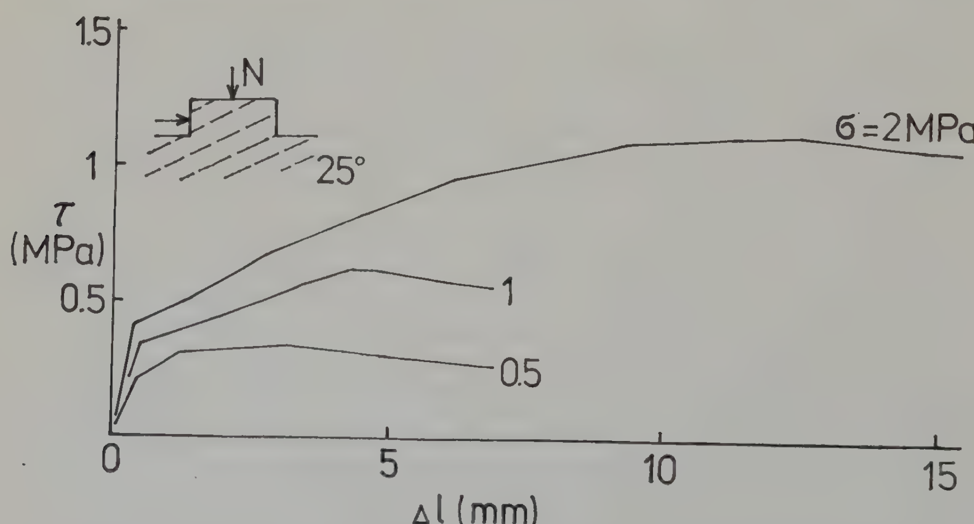


Fig. 6–21. Results of block shear tests on saturated hard fissile claystone, Liptovská Mara, Slovakia. Maximum resistances yield $c' = 0.07$ MPA and $\varphi_t = 27°$. The large increase in block displacement Δl indicates ductile behaviour of the block under a normal loading of 2 MPa (Mencl and Trávníček 1965).

The unfavourable effects of the volume decrease associated with the change in the behaviour of the material from brittle to ductile was noted in sections 4.3 and 4.6, but the knowledge of the degree of compression at which the change occurs is scanty. Field shear tests should, therefore, be aimed at gaining more information on the threshold value at which rigid behaviour changes into ductile behaviour (Fig. 6–21). It should also be remembered that this threshold and the extent of the volume changes are both dependent on the angle between the direction of the shear force and the planes of greatest weakness.

The results of field tests are interpreted in conjuction with the results of geological surveying and the results of laboratory tests. This topic will be discussed in section 6.3.

6.1.8.3 Determination of the density of sandy soils

In the case of sandy soil, importance is attached to the density and permeability of the soil. The spontaneous onset of liquid behaviour in saturated fine sands is an extreme example of failure caused by low density and low permeability of the soil. The density of sandy soils can be determined by field penetration tests.

6.1.9 Measurement of residual horizontal stress

The importance of the general state of stress of a soil or rock mass in the investigation of sliding movements has been emphasized in several places in this book. The possibility of the existence of the horizontal pressure which exceeds the value of the earth pressure at rest (as determined by the actual overburden pressure) was referred to. The unfavourable influence of such horizontal stress on the stability of the slopes of excavations was also mentioned. Consequently, there is a need for thorough investigation.

Most of the existing methods of investigation involve the "removal of material" technique, which was first used by Sachs (In: Dow 1965) for metals. It is based on the fact that the stress field of a strained body changes after a part of the body is taken away. This induces a measurable deformation of the body. From the value of the altered stress field, from the measured deformation and from the stress-strain parameters of the material, the original stress field can be computed.

As the deformation modulus of rocks differs considerably from that of soils, the methods of measurement differ for the two materials. As far as rocks are concerned, the method is one that was developed in ore mines, and later on it was also used in engineering geological investigations. Measurements of the deformation of boreholes provide suitable data for analysing the stability of slopes. A cylindrical measuring device (a "stressmeter" or "borehole gauge") is inserted into the borehole to a given depth. Any deformations of the cross-section of the borehole are transmitted by means of the sensitive blades of the device to electrical resistance strain gauges wired to the surface. Deformation can be produced by several "*removal of material*" techniques. For example, pressure on the rock of a borehole wall may be relieved by concentric overcoring with a drilling machine fitted with a diamond drill bit of larger diameter. Thus the rock around the former borehole is removed. The release of presure deforms the overcored cylinder of rock and the deformation is measured. The success of this procedure depends both on the construction of the stressmeter, and on the accuracy with which the borehole is overcored.

Of the many methods that have been used, let us quote those of Hast and Postgraduate School of Mining, Sheffield. The method of Hast (1958) involves the translation of mechanical deformation into electromagnetic impulses (magnetostriction). In the method of the Postgraduate School of Mining (Roberts et al. 1964) a ring of photoelastic sensitive material is inserted into the borehole, and the photoelastic changes which are produced by any deformation of the cross-section of the borehole are observed from the opening. The advantage of the latter method is that knowledge of the deformation modulus of the rock is not necessary.

The need to overcore the stressmeter borehole restricts the method to boreholes of limited depth; a depth of 10 m can be regarded as the usual limit, but depths of 20 m have been overcored successfully. These are the depths at which the residual stress occurs. Greater depths can be reached by sinking the boreholes from shafts or galleries.

The measurement of residual stress can reveal "dead rocks", that is, rock bodies in which the residual prestrain has been released, for example by erosion of an adjacent valley.

Another method applicable to weak rocks and stiff soils was used by Mencl (1965b) to investigate a cutting near Bánovce (Fig. 4–12). The bottom of the cutting was excavated in flysch shales. Whereas practically no pressure was registered in the material squeezed up by the landslide, measurements in the intact section of the cutting indicated a pressure of 0.4 to 0.6 MPa. A hole 30–40 cm deep was bored by means of a tube with a serrated lip. At the same time, the displacements of four points set around the borehole were measured. The most suitable diameter for the borehole was found to be 15 cm, while the distance of the displacement points from the centre of the borehole was 33 cm (Fig. 6–22).

The immediate deformation modulus E_m enters into the formula

$$\sigma = \Delta r \,.\, r \,.\, E_m/[(1 + \nu)a^2]\,,$$

in which Δr is the displacement of the observed points at distance r from the centre (σ is the compression if the displacement Δr is toward the axis), a is the borehole radius, and ν is Poisson's ratio.

The formula has been determined from the elastic solution. The finite element analysis, if based on the pseudoelastic approach, yields smaller σ because the actual

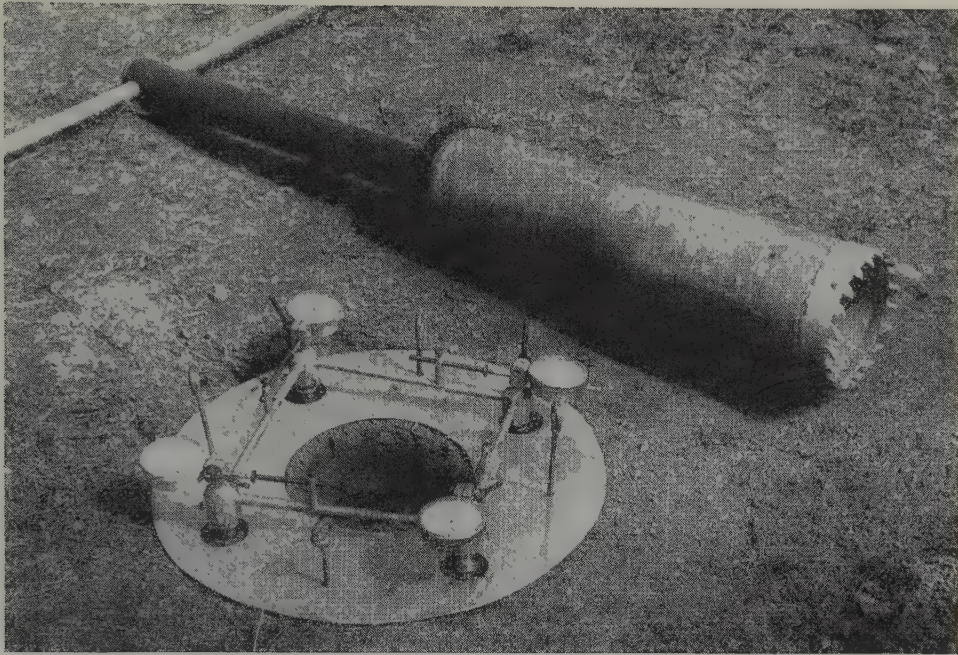


Fig. 6–22. Measurement of stress in the surface layer of a landslide (Mencl 1962).

E_m is smaller than the basic value of E_m. Owing to the change in the stress field around the borehole the normal stress decreases in the radial direction but increases in the tangential direction. This decreases the factor of safety and therefore E_m also decreases. It can be said that the magnitude of a half of the E_m value valid for the all-round compression loading appears to be a suitable quantity.

If the displacements differ in two perpendicular directions (in the direction of greatest steepness and the direction of the contours), the compressions in the two directions can be denoted by σ_1, and σ_2, and the corresponding displacements at the points of measurement by Δr_1, and Δr_2. Then the elastic solution yields the formulae

$$\Delta r_1 = \sigma_1 a^2[5 + \nu - a^2(1 + \nu)/r^2]/2E_m r + \sigma_2 a^2[-3 + \nu + a^2(1 + \nu)/r^2]/2E_m r\,,$$

$$\Delta r_2 = \sigma_1 a^2[-3 + \nu + a^2(1 + \nu)/r^2]/2E_m r + \sigma_2 a^2[5 + \nu - a^2(1 + \nu)/r^2/2E_m r\,.$$

As the test is simple to carry out, it can be repeated several times, and at several depths of excavation. A large area may be covered in one day. The possibility of horizontal forces developing as a result of volume changes accompanying desiccation or swelling must be considered. It is better not to perform the tests on the ground, and if possible at least a few decimeters of the soil profile should be removed.

The method is helpful in distinguishing between the main parts of the landslide area, e. g. between the head area and the area of deposition. For example, the tests have provided confirmation of the existence of a tension area on a slope surface as predicted by static analysis (Fig. 7–6). Interesting test results were obtained by Fussgänger and Jadroň (1977). They took measurements along profiles of the landslide in the Palaeogene basin of Liptov in Slovakia (profile L in Fig. 6–23). The results

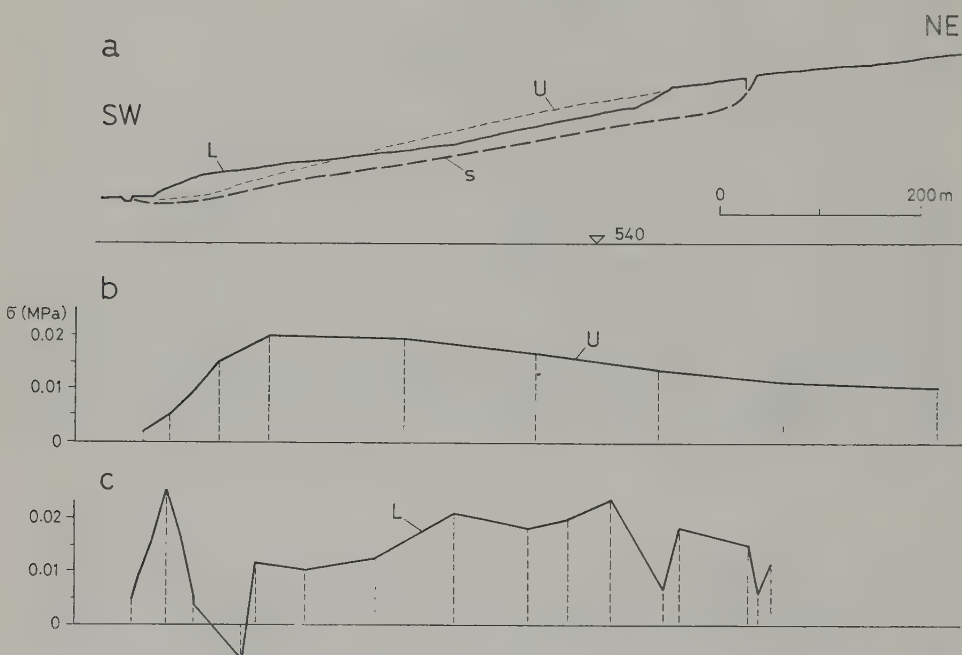


Fig. 6–23. Profiles of the slope at Okoličná, Slovakia. U — profile of the undisturbed flank of the landslide, L — profile of the slide area, s — slip surface. The results of stress measurements along these profiles are plotted in (b) and (c).

were checked against stress measurements across the undisturbed slope nearby (profile U). Although the slope movement is in a state of slow creep, it is nevertheless threatening to disrupt an important railway line. The compression at the toe of the slope is evident from profile U, and the build-up of compression near the mid-height of the sliding slope in profile L indicates the transition from the active to the passive sections of the sliding mass. A local slip near the toe is highlighted by the appearance of both tension and compression stresses.

Recently, Marchetti (1979) has developed a flat dilatometer which consists of a steel blade carrying on one side a thin flat circular expandable steel membrane, 60 mm in diameter. The blade is jacked into the ground and the membrane is inflated by gas under pressure. It is interesting that the coefficient of lateral stress K_0 was found to be higher than might be expected, e. g. of the order of 2 in glacial clay.

6.2 Geophysical methods

As in other fields of engineering-geological activity the classical geophysical methods may supplement the results of direct investigation of the site. However, the peculiar problems of slope movements have led to the development of specialized techniques of exploration.

The presence of planes of weakness, and the nature of ground-water movement and its provenance, are the foremost matters of interest to those who have to analyze the safety of a slope, or decide on what measures to take in order to increase its stability.

Location of the slip surface, the evaluation of the degree of straining, and establishing the course of ground-water movements, are the chief concerns in dealing with existing natural or man-made slope movements. The presence of weak surfaces or weak bands (weak layers, faults, or loosened rock masses) can be traced by electrical resistivity and by seismic methods; the weak material displays lower resistivity and a lower velocity of transmission of seismic waves. Because the presence of ground-water can interfere with measurements, it is in the higher reaches of the slope that the resistivity method is most promising. The depth of the ground-water level can be traced by the resistivity method. In many rocks, e. g. in the Neogene clays, the high salinity of ground-water produces a very distinct drop in the apparent resistivity.

Also, the tracing of vertical water movement in boreholes is an important task, because it reveals changes in the ground-water table at different depths as well as the position of the most permeable layer. Temperature measurement is one of the methods used to tackle this problem, and another approach involves measuring the degree of dilution of soluble substances which have been added to the water in the borehole. Ground-water percolating into the borehole dilutes any solutes in the water of the borehole, starting at the point of entry. Any upward or downward movement of the diluted water layer indicates where a lower piezometer head exists, and points to the amount of variation in the ground-water levels of individual soil layers. Different techniques may be used to estimate the strength of the solution, including radioactivity measurements where radioisotopes are used.

Investigation of the source of ground-water is important, particularly if subsurface drainage of the slope is planned. However, if the water is issuing from the bedrock under a covering of less permeable soils, and is flowing out over an extended zone (e. g. a fault), such a zone is not easy to locate. This is the situation shown in Fig. 13–2. A similar difficulty is often encountered in connection with the drainage of Late Tertiary slopes. A case was investigated by Yamagushi (1977) using the electric resistivity method. A map of apparent resistivity was compiled for the area, and the places of lowest resistivity were specified as locations for pumping wells. A similar approach was referred to by Tochiki (1977), who established the presence of faults and crushed rocks by measuring natural radioactivity as well as electric resistivity in the unstable areas of the steep slopes of the main tectonic zone in Japan.

Locating the positions of developing slip surfaces is also of considerable importance. Direct location from classical geophysical measurements is not easy, and is based largely on the fact that some loosening of the rock occurs along the slip surface owing to dilatancy.

Belyj et al. (1970) reported an 8 to 10% reduction in the work of investigating the slope of reservoirs when microseismic measurements were used. The differences in the wave velocities at two observation points can be used to arrive at an estimate of the depth of the slide body, or even its differential straining.

For several years both the position of the slip surface and the rate of slope movement have been studied by the *geoacoustic method*, first applied by Obert and Duvall (in Stateham and Merill, 1979) to predict instability in underground mines, and to the study of landslides by Cadmann et al. (in Novosad 1977). Rock noise is scanned by sensitive electrodynamic geophones and is recorded on tape. The records are then evaluated with oscillographs. Because other sources of acoustic noise are present in soils and rocks, a problem of interpretation arises. Phrases as "the state of potential danger" which sometimes appear in the reports may be unduly alarming. The analysis of the frequency of the acoustic events becomes to be preferred by the experts. For example, Stateham and Merill (1979), who studied the problems of stability of slopes in four ore open pits, indicate the following correlations: 0–10 noises per hour: stable slope in an inactive mining area, 10–50 noises per hour: stable slope in an active mining area, and more than 50 noises per hour: unstable slope.

The noise level decreases very rapidly with distance from the source in clayey soils, and therefore the noise level indicates not only the degree of the factor of safety but also the depth of the sliding surface.

6.3 Laboratory investigations

There is little need to emphasize the importance of laboratory investigation of soil and rock samples from sites of slope movement. On the other hand, an important observation has to be made, namely that the laboratory is often remote from the slope site, not only physically but also on an intellectual level, so that there is always a danger of laboratory work and field work failing to complement one another. Laboratory tests often reveal a need for further field studies which cannot be undertaken because the programme of field exploration has been wound up. A more banal situation is that in which a lack of undisturbed samples becomes evident. Therefore, at least a part of the laboratory investigation should run parallel with the field work and should follow a schedule drawn up by the engineering-geologist responsible for the overall problem.

A purposeful discussion of individual results of laboratory testing is given in the following subsections.

6.3.1 *Mineralogical composition*

This is a large and complex subject, and therefore only some of the factors related to landslides are briefly discussed here.

(a) Under similar conditions, those cohesive soils containing minerals of greater activity are the more hazardous. Claystones and clayey shales with layers of montmorillonitic clay are particularly troublesome. The swelling of the clay layers is followed by the development of tensile cracks in the rather stiffer beds of shales. The cracks develop very close to one another and the rock changes into a mass with the characteristics of a fissured clay. Probably the smallest strength parameters found were those reported by Underwood (1964) on the Harlan project (a tributary of the Missouri river). In field tests, Cretaceous marlstone with interbeds containing 80% montmorillonite exhibited a strength of $\tau = 0.015 + \sigma \tan 7°30'$ (MPa). The presence of volcanic products in the wider environment of a site should be always a warning signal that a low shear strength of clayey soils could be expected (Fig. 5−37).

(b) The effect of bound cations on the behaviour of slope soil is influenced by three factors:

Firstly, the soil may contain bound water alone, or there may be free water present as well. In the former case the accepted hypothesis that the cations of small radius and high valency produce a high shear strength holds true; the mechanism of this is associated with the distances between particles. Kazda (1961) found that when the water content was 10% (a water content of materials between claystones and shales) the strength of the calcium form of the clay was several times greater than that of the sodium form. However, if free water is also present, the calcium form is not capable of holding as much water as the sodium form, as shown by the tendency to slaking. As early as 1947 Bernatzik pointed out that clay weakened on contact with a cement grouting mixture.

Another factor of importance in the occurrence of landslides is the remoulding of the soil mass. The behaviour of remoulded illitic clays differs from that of montmorillonitic clays; an increase of the potassium form is associated with greater strength in illitic clays, both in the undisturbed as well as the remoulded state. On the other hand, potassium ions give rise to a closer arrangement of the elementary layers in montmorillonitic clays, which results in the expulsion of some of the free water, thus bringing about a decrease in strength upon remoulding. This is of practical importance near the surface of slopes, as potassium ions released by weathering processes increase the strength of illitic clays, but can have an unfavourable effect on montmorillonitic clays.

The kind of cations can also affect the rate of consolidation of clays. A higher valency and a smaller diameter of the hydrated ion increase not only the strength but also the permeability of the material. On the other hand, the permeability of sodium forms is small. Anions also increase permeability, and therefore have a bearing on the effectiveness of drainage trenches on slopes consisting of fissured clays. Slag enriches the soil with anions and the permeability increases.

(c) The third problem concerns the weathering of clayey soils, by which the iron compounds are affected. On oxidation, the crystal lattice of the clay minerals releases cations, especially potassium. This process was observed in the upper section (i. e. the earthflow) of the landslide at Handlová.

(d) Potassium ions may be transferred from overlying beds. The susceptibility to sliding of glauconite-bearing sandstones of the Bohemian Cretaceous formation may be explained by the release of potassium from the glauconite into seepage water by ion-exchange. This brings about the dispersion of subgrade clays and transforms them into a suspension which is carried away by seepage water.

(e) The recent findings of Massarsch (1979) are of great importance for the problems of slope movements of clays. From studies carried out by the Swedish Geotechnical Institute, the author shows that the sensitivity of soft clays increases when they are infiltrated by waste water. Laboratory tests have demonstrated that non-sensitive clays may be changed into quick-clays by the infiltration of certain detergents and organic substances.

6.3.2 Index properties of soils

Information on the simple physical properties of soils is important in the analysis of the static behaviour of slopes. This is particularly so in the case of clayey soils, because the shear deformation moduli as well as the shear strength of these soils change considerably with water content. Thus, for example, an increase of about 1% in the water content of a stiff plastic Neogene clay (with an illitic clay content) produces a decrease in shear strength of about 15 %.

Such an increase in the water content may follow from a volume increase, as often occurs along slip surfaces (subsect. 4.3). In stiff, plastic Neogene clay this increase is of the order of 3 to 4%, making it possible to identify the position of the slip surface provided that continuous and undisturbed samples can be obtained from the boreholes. Therefore those responsible for the static investigation of a slope first of all examine the water content data wherever clay is concerned. The water content figures are evaluated in conjunction with the plastic limit of the material, and in this way the first clues as to the future behaviour of the slope are obtained.

The Atterberg limits have proved to be a good indicator of the nature of clayey soils, and therefore the determination of the Atterberg limits and evaluation of their significance should always be considered. When a landslide occurs in surface layers, the change in the Atterberg limits should be followed down the depth of the boreholes and the values obtained should be correlated with the mechanical properties of the soil. But it must not be forgotten that the Atterberg limits are determined using specimens of remoulded soil, and, in the case of montmorillonitic clays, the cations of higher valency which are often present contribute to a reduction in the liquid limit, since water is expelled from the lattice structure. On the other hand, clays exhibiting no adsorption capacity of the crystal lattice (as, for instance, kaolinitic and illitic clays) behave in the opposite way: Na-illite has a low liquid limit, and the K^+, Mg^{2+}, Ca^{2+} and Fe^{3+} forms show successively higher liquid limits (Kazda, 1961b). The same sequence is encountered in relation to the plastic limit.

There is no doubt that the values of the Atterberg limits are associated with the mineralogical composition of the clay fraction. The latter is studied by means of chemical, X-ray and differential thermal analyses. Since the clay fraction may be present in a larger or smaller proportion, the Atterberg limits are also affected by the amounts of clay particles. Therefore, values for the Atterberg limits cannot be directly correlated with the mineralogical composition without considering the proportion of the clay fraction also. This leads to the evaluation of the so-called activity of clays, i. e. the plasticity index expressed as a ratio with the percentage (by weight) of particles smaller than 0.002 mm. The activity is universally quoted in reports, but care is necessary in the determination of the percentage of particles smaller than 0.002 mm. Measurements of the latter are frequently invalidated on account of coagulation. Since activity values are usually compared with those of Skempton (1948, 1953) based on British standard methods of size analysis, these standard methods should be employed, especially as regards the nature of the dispersing agents used to prevent coagulation.

6.3.3 Rate of consolidation under compression

Several stability problems, namely those that arise when the weight of an embankment is being used to increase the strength of the underlying soil by consolidation, require the rate of consolidation to be assessed. For example, a supporting fill may be used to stabilize the substratum of planned embankment (Fig. 6–24). In this case it

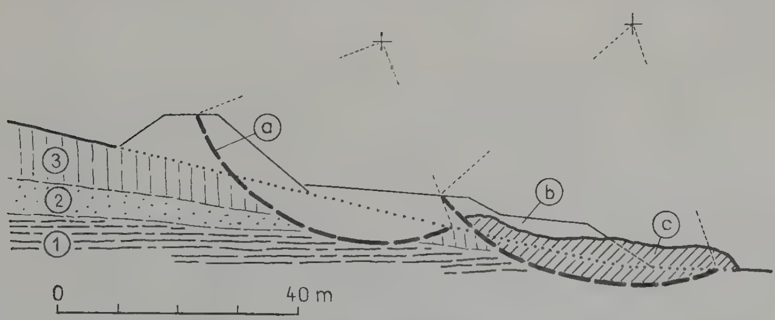


Fig. 6–24. Landslide (a) involving the substratum was caused by the weight of a road embankment. An attempt to stabilize it with a supporting fill (b) failed, because this provoked further sliding (c); 1 — Neogene clay, 2 — glaciofluvial sand, 3 — loess loam.

is necessary to know whether or not consolidation of the underlying soil will be achieved in a reasonable time. Consolidation may be accelerated by driving consolidation sand piles or similar elements into the subsoil; in the majority of cases (i. e. with the exception of non-fissured, highly plastic clays), most of the consolidation process takes place during the construction period.

The time of consolidation can be investigated by oedometer tests on undisturbed samples in the laboratory. However, detailed studies of the settlement process have indicated that this method gives a consolidation rate many times smaller than the actual rate, and therefore the permeability tests are recommended instead, preferably using in-situ permeability tests (Schlosser, 1978). One of the main reasons for this state of affairs is the anisotropy of soils; the horizontal coefficient of permeability is usually greater than the vertical coefficient, and it is the latter which corresponds to the data obtained in oedometer tests.

The efficiency of vertical drains built below embankments depends on the magnitude of the horizontal permeability. An example is shown in Fig. 6–25, which gives the rates of settlement under the embankments of a highway and a branch road. Sand piles 9 m long were installed below the main embankment, but none were placed below the smaller one. Settlement of the subsoil ceased after the main embank-

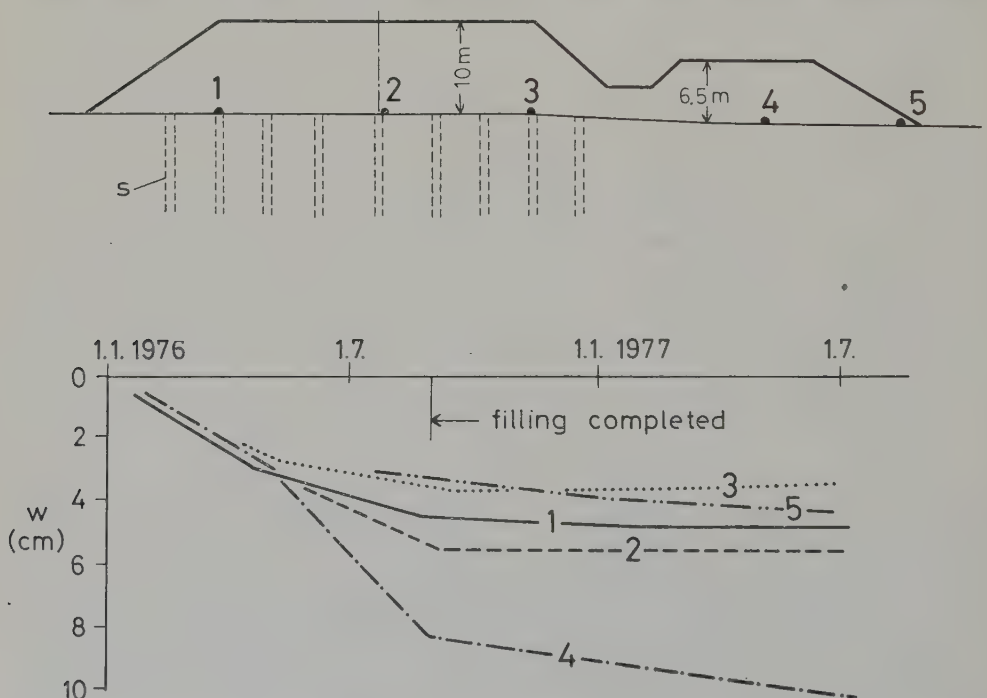


Fig. 6–25. Settlement of the Oligocene marl (average $w = 26\%$ $w_p = 23\%$, $w_L = 51\%$) which forms the substratum under the embankment of a highway and a branch road. Sand piles (S) were driven into the marl in order to accelerate settlement (Pešina 1978).

ment has been filled, a development that did not occur in the case of the smaller embankment. It may also be noted how much less the main embankment subsided compared with the smaller one.

6.3.4 Stress-strain parameters

As already shown in Chapter 4, and as will be seen in the analysis of stability in Chapter 7, problems of slope safety are of a statically indeterminate character. This means that the deformation conditions have to be considered in the static solutions. The deformation characteristics of rocks and soils have therefore become important tools in the processes of static analyses.

The solution can be arrived at in either of two ways, as follows.

(1) Modelling the progressive development of the slope. For example, erosion, excavation, or some other agent may have caused a part of the ancient ground to be removed. This process is statically simulated by introducing the negative contact forces at the boundary between the removed and remaining areas of the ground. This technique works well in the static analysis of excavated slopes, and was used to obtain the solutions shown in Figs. 4–1 to 4–11 (except Fig. 4–5).

(2) Assuming that the present condition of the slope is without internal and external forces, and then loading it, step by step with gravity forces, hydrodynamic forces, etc. Although not correct from a theoretical point of view, this approach seems fruitful in the case of natural slopes, and the solutions presented in Figs. 4–16 and 4–17, as well as those in Figs. 7–6, 7–8, 7–9 and 7–10 were obtained by this method. Practical experience in applying the foregoing method to natural slopes has not been favourable. Probably the interplay of forces during the formation of the slope produced a more complicated situation than that represented by a removal of forces. Furthermore, rock properties changed during the sculpturing of the relief.

By means of either of these methods the effect of any additional loading (e. g. a 5% increase in the overall weight) can be determined, and from this the safety of the slope assessed. This approach will be dealt with in section 7.7.

With respect to the former of the two methods, it is clear that the rock mass is assumed to have been compressed by gravity forces greater than those in existence after modelling of the slope. Therefore the bulk deformation characteristics required for the former of the two methods are predominantly related to the unloading of the normal stresses. On the other hand, the shear forces have increased during modelling of the slope. Therefore, when a soil or rock is represented by its deformation parameters, it is important that these parameters are based on the unloading process.

In the second method, the present quality of the rock is assumed to be representative of the virgin state, and both the compression and shear stresses increase.

The process of deformation in rocks and soils involves elastic and plastic components. The pseudo-elastic approach is followed in the present treatise. This means that the relationships between stresses and strains (unit deformations) are formulated using the elasticity laws. This is not without its dangers, but if appropriate precautions are taken, it gives reasonably good results, and is currently being used by many workers. In principle, it gives deformation moduli that increase with three-dimensional compression, and decrease with shear stressing of the material, whether it is being loaded or unloaded.

The pseudo-elastic deformation moduli pertaining to the primary loading of rocks are not easy to determine. Fortunately, the high degree of accuracy is important only where weak rocks are concerned. The results of plate loading tests of primary loading can be accepted over a range of low safety factors. The so-called modified core recovery ratios[1] provide a basis for determining the moduli for higher safety factors.

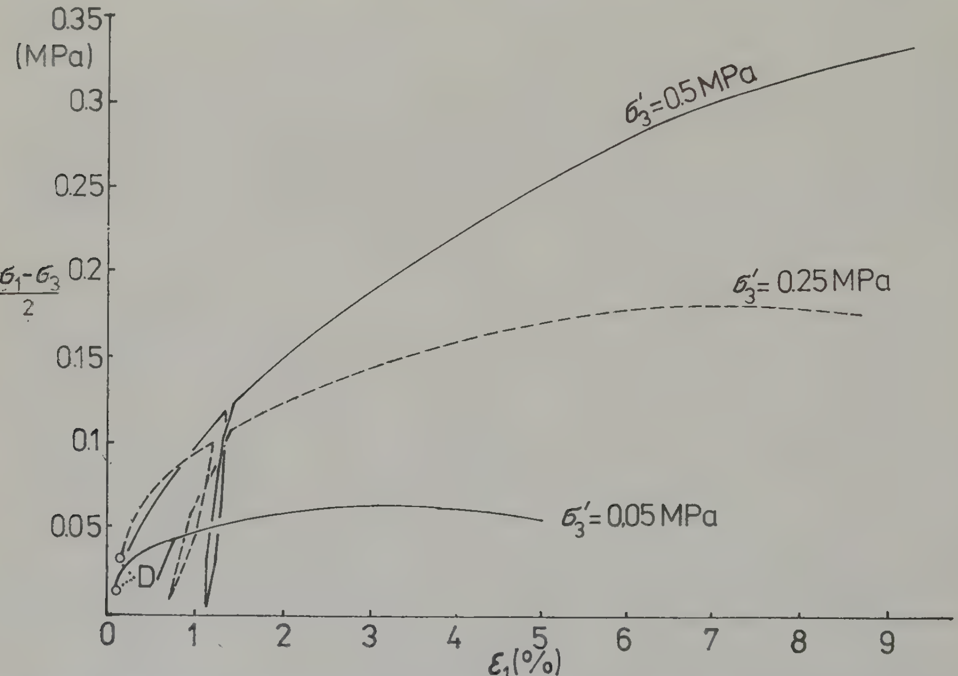


Fig. 6-26. Stress-strain characteristics of a stiff Neogene clay (Králová, Slovakia); $w = 24\%$, $w_p = 23\%$, depth is 24 m below ground surface and 5 m below river gravel; D — onset of dilatancy; the considerable strain at a confining stress of 0.5 MPa indicates ductile behaviour (free interpretation of the tests made by Havlíček, 1974).

[1] This index property is a measure of the degree of jointing. The total sum of the core lengths as a percentage of the corresponding length of the borehole is established. Careful diamond drilling is necessary and only core lengths greater than 10 cm are considered for the calculation (the shorter are included only when it is clear that they are broken pieces from the longer cores). The result serves as an indicator of the ratio E/E_d, where E is the unknown deformation modulus and E_d is the dynamic modulus found by geophysical exploration. This approach was introduced by Deere et al. (1966), who also developed the concept of reduction indices.

The deformation moduli for unloading can be obtained in a similar way, the unloading part of the loading tests being applied to the investigation of moduli associated with low safety factors; the dynamic moduli apply to the higher safety factors.

The loading and unloading deformation moduli of clay can be determined from the results of laboratory triaxial tests. There is some doubt concerning the accuracy of this kind of test, but it is the relationship between values obtained under different degrees of compression and shear stresses that is more important in the numerical analysis of the safety of slopes than the absolute magnitudes of the moduli. From this type of test, diagrams such as those shown in Figs. 6–26 and 6–27 are obtained. It is desirable to construct diagrams of the latter type for each rock or soil occurring in the analysis. The following points should be noted:

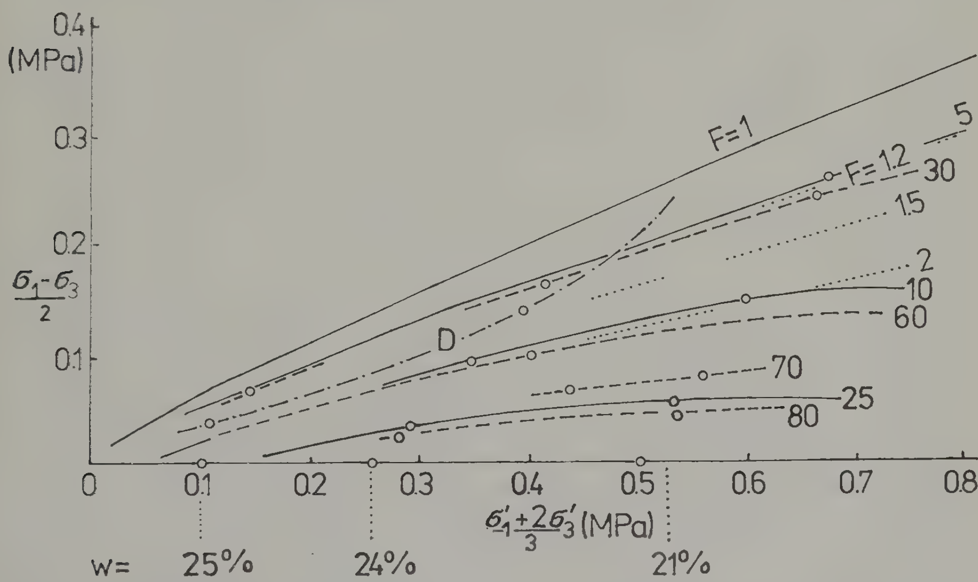


Fig. 6–27. Pseudo-elastic deformation moduli (MPa) for primary loading (full lines) and unloading (dashed lines) in the field of safety factors (dotted lines), for the clay in Fig. 6-26. The intensity of the compression stresses is represented by abscissae, and the intensity of shear stresses by the ordinates. Line D indicates the onset of dilatancy; w = water content at different compression stresses.

(a) The changes in the magnitudes of the moduli along the axis of the abscissae represent the influence of compression or decompression. However, both the increased compression and increased quality at greater depth exert a modifying influence on the values of the moduli. This prompts the question of whether the same relative values are obtained when only one of these factors is operating. Often it is better to sort separate zones of equal rock quality (e. g. according to the degree of weathering), and to construct separate diagrams for each of them.

(b) A factor which should be noted (particularly in connection with the mathematical aspect of the diagram) is the rapid decrease in the moduli as the shear stresses increase and the sphere of ductile behaviour of the material is entered (the right hand section of the diagram of Fig. 6–27).

(c) A volume increase in a triaxial test is evidence of the brittle behaviour of the material. The states of stress at which this begins to occur are denoted by D in the diagram, and should be recorded.

6.3.5 Shear strength

Problems connected with the shear strength of soils and rocks have received a lot of attention elsewhere, and therefore only those of special relevance to the analyses of slopes will be discussed here.

(a) Shear strength of clayey soils under normal pressure. As shown in Chapter 4, the slope foot is highly stressed and its local safety factors are low. The magnitude of

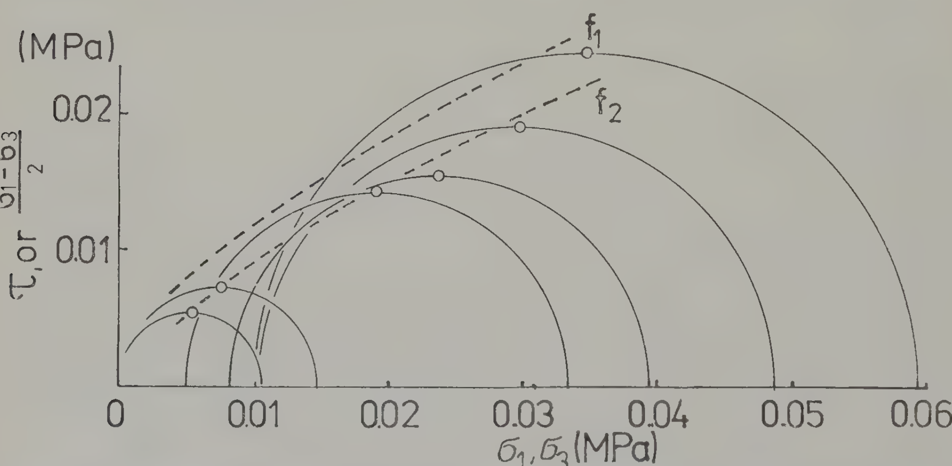


Fig. 6–28. Shear strength of plastic clay ($w_p = 30-35\%$ $w_L = 56-66\%$) represented by the Mohr envelope ($\mathbf{f}_1$) and by the maximum shear stress ($\mathbf{f}_2$) (after Iyer, 1975).

its resistance to failure is therefore important. Compressive stresses are small owing to the very small overburden, and this is still further accentuated where ground-water is flowing out of from the slope. The effect of swelling on the cohesion component of the effective shear strength may be unfavourable appearing as a bending of the strength envelope near the origin of the coordinates in the Mohr diagram. Although several references to this can be found in a number of treatises, the results of tests carried out by Iyer (1975) under low normal pressures provided first elucidation of the problem (Fig. 6–28). In areas where frost is an important factor, e. g. in Central Europe, this decrease in shear strength may also serve as a measure of the degradation of the soil structure after thawing. Such a decrease has been incorporated into the formulation of the strength parameters applied in the static solutions in Chapter 4. In this way also the effectiveness of the stabilization embankments filled at the toe of the slopes turned out to be greater than the classical solutions seemed to indicate (section 8.2).

(b) Long-term shear strength of soils and rocks. Treatises on the problems of creep movements are numerous, and there is little to add for the present. However, several remarks are appropriate:

– Firstly, meanings of the term "creep" as it is used in engineering geology and mechanics differs. Whereas it refers to a slow movement in engineering-geological terminology (the velocity is not exactly defined, approximately a few centimetres a year), in mechanics it is used to refer to a steady, slow displacement under constant stress without any interference from external agents. Many slow displacements which develop in the impermeable and saturated rocks result from a high pore pressure associated with a reduction in volume. These conditions are caused by the ductile (contractant) behaviour of the rock. The volume decrease is not therefore brought about by the pressure of the overburden, but is caused instead by small shear strains which affect the slope. The increase in the pore water pressure reduces the effective shear resistance of the material. Many creep movements (in the sense of the geological terminology) belong to this category and are not creep movements in the mechanical sense. The importance of this distinction becomes apparent when remedial measures are under consideration. Creep movements produced by an increase in pore water pressure can be stopped, for example, by installing consolidation sand piles as is often done at the base of high embankments. This technique would fail to solve the problem where true creep was concerned.

True creep shear displacements of soil and rock masses occur within a stress range of about 75 to 100% of the short-term shear strength. There is probably one exception to this rule and that is the shear strength of clay interlayers along the brown coal seams. This will be discussed in section 13.3.1.

– Many high rock slopes show evidence of a deep creep movement although their shear stressing is less than the long-term shear strength. Let us consider in more detail the case shown in Fig. 5 – 60. The topography of the slope and the shape of this shear zone suggest that the shear strength angle is about 24°. The phyllite and gneiss mass is sheared across the schistosity planes and the direction of the principal stress is almost parallel to the schistosity planes. The rock displays a shear strength angle of 36°. This difference between the two magnitudes of the shear strength angle is too great to be explained by a stress of 75% of the shear strength. Probably the topography of the slope or the hydrogeological and temperature conditions prevailing when the slope movement and disturbance of the rock structure began, were different from the present-day conditions.

(c) Residual resistance of clay. The presence of joints in a clay mass – a characteristic of fissured clay, requires application of the residual shear resistance to the design of slopes. Skempton (1970) analyzed the problem and showed that the angle corresponding to slope safety is about 4° greater than the angle of the residual shear strength in unweathered London Clay. The case of a slope 95 m high in a typical fissured clay will be referred to in sect. 7.9; the fully drained slope has remained undisturbed for many years at an angle of 16°. Laboratory tests gave a residual shear angle of about 10°.

Divergences also occur when the residual strength along existing slip surfaces is investigated. Back calculations give shear angles about 2 to 3° greater than those obtained by laboratory tests (Mencl et al. 1977). Hutchinson (1977) indicates that differences of this kind are frequently encountered and some caution is probably advisable when applying results for clays of very high plasticity. The case of a road embankment failure will be discussed in section 11.4 (Fig. 11–16); the clay material forming the base displayed a liquid limit of about 90%. The reliable back calculation gave a residual strength angle of 10°, a value which agrees well with the diagram of Jamiolkowski and Pasqualini (in Cancelli 1977).

(d) Relaxation of shear resistance is another phenomenon which is relevant to the stability of slopes. When a sliding or creeping earthen mass bears against a pile wall or similar fixed structure, the shear strength of the mass as a proportion of the total resistance is smaller than expected. On the other hand, the pressure on the wall is greater than calculated from the long-term resistance of the material. What happens is not clear, and the question remains as to what extent the deformation properties of the mass are involved in the process. For practical purposes, however, the decrease in shear strength must be taken into consideration in the design of fixed retaining structures. According to the results carried out by Mencl and Trávníček (1964), the shear resistance of clay decreased to the level of 30% of the classical short-time peak value. On the other hand, no loss of shear resistance could be detected in the hard and unsaturated fissile Palaeogene claystone.

Chapter 7

STABILITY ANALYSES

Although experience as well as a good understanding of site conditions from the engineering-geological point of view are of primary importance when investigating the possibility of failures in natural or artificial slopes, there is nevertheless a need for exact static investigation. Ever since large-scale excavation works were undertaken for canals and railways, methods of static analysis have been sought and developed. Because the theory has had to meet with real situations, this development has gone hand in hand with an expansion of work in both the field and the laboratory as refinement of the static analysis has taken place. The present chapter deals with several stability analyses, emphasis beeing placed more on an evaluation of the adequacy of different approaches than on a description of the many existing methods.

7.1 Preliminary analysis during exploratory work

Soon after the laboratory work has begun the stability of the slope should be assessed. This is often neglected, the stability analysis being postponed until the exploration has been completed. If the analysis is undertaken sooner, then questions which arise out of the analysis can be properly investigated as a part of the programme of exploratory work. For example, the preliminary stability analysis which is based largely on the first results of the investigation, may indicate that a slope is stable although some initial movement has already occurred. If the slope consists of Tertiary claystones laid-down on a gneiss bedrock, there is a possibility of concentrated uplift forces developing in connection with a possible fault not recognized in the investigation so far. The opposite type of situation is also not rare, e. g. the slope may remain stable although the preliminary analysis has indicated that it should not. Have the strengths of materials not been underestimated, and should field tests not be incorporated into later stages of the investigation?

7.2 Method of analogy

In many cases the elements of static analysis are based on concepts derived from direct experience. The importance of experience was emphasized by Ch. Terzaghi and R. B. Peck in Art. 49 of their "Soil Mechanics in Engineering Practice" (1948), in which they discuss a compromise between the requirements of economy and interests of slope safety. This approach by analogy may be applied either to local or to regional situations.

Confidence placed in broad local experience has brought good results in open pit mining. An interesting example was presented by Coates et al. (1979), who analysed what was chosen as the most economic slope, very near to a critical one, in an open ore mine of 335 m depth in Canada. The rock was of pyroclastic origin, partly altered and tectonized, exhibiting an angle of shear resistance of $35 \pm 7°$. Two previous slides were taken into consideration and an overall slope angle of 36.5° was adopted. Apart from one minor slope failure, an important slide occurred one month after the mining operations had been finished.

In open pit coal mining the role of local experience is of great importance, and this will be discussed further in Chapter 13.

Regional experience in the design of slopes has been most widely relied upon in the construction of roads and railways. A recent example of this is the construction of an express highway in the Neogene clayey deposits of the Carpathian foredeep in Czechoslovakia. Reliance upon experience is born of the uncertainties in the static analysis of slopes, as will be discussed in the following paragraphs. A similar situation occurs in many cases of rock slopes.

7.3 Use of graphs

Early in the development of the static analysis of slopes, there was a tendency to compile design charts for the stability of slopes as a labour-saving aid to designers. Several such charts, for example those of Morgenstern and Price (1965), have been based on progressive methods of classic analysis (Bishop 1955). Their use is of great assistance to designers provided that several factors not considered in the charts (these factors are discussed in Chapter 4 and in the following paragraphs of the present Chapter), have been taken into account in choosing the magnitude of the safety factors or other corrective coefficients.

7.4 Numerical analyses

Forces and other elements of analysis. Forces arising from the weight of the soil or rock mass do not present much difficulty, as a simple resolution or composition of forces is all that is necessary to perform the classic analyses of slopes (Section 7.5).

On the other hand, forces arising from seepage or water pressure require explanation, especially as there are two alternative ways of introducing them into the static analysis.

A useful approach in both classic analysis and the finite element method, is to take these forces as external forces acting on the walls of the imaginary separation surfaces which are considered for the purposes of analysis. Then the forces are simply identical with hydrostatic pressure, and the latter can be determined if the ground-water level corresponding to the appropriate point is known.

For example, if the surface as shown in Fig. 7–1 is the slip surface, the hydrostatic pressure (the term uplift is commonly used when the hydrostatic pressure acts under the base of a body) is a force normal to the slip surface and the ordinate of the loading

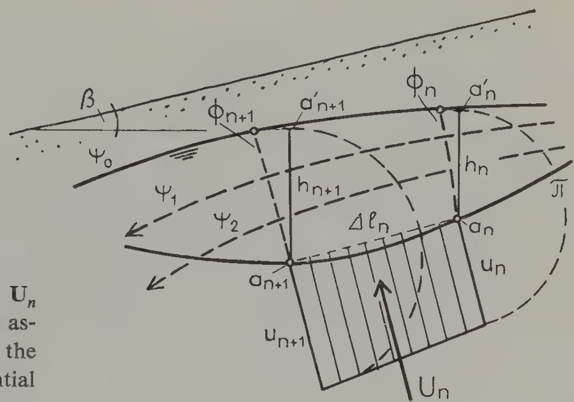


Fig. 7–1. Loading diagram for uplift $\mathbf{U}_n$ acting in the sector a_n to a_{n+1} of the assumed slip surface. ψ_0, ψ_1 and ψ_2 are the flow lines; ϕ_n and ϕ_{n+1} are equipotential lines of the ground-water flow net.

diagram corresponds with the piezometer head. The task of estimating the piezometer head is a little more complicated than indicating the hydrostatic pressure of free water. When water flows through the soil its piezometer head at any particular point is not the same as the vertical distance between the point and the ground-water level. It is necessary to make a draw of the water flow net, albeit with some degree of approximation, and to consider the equipotential line which intersects the point in section.

Fig. 7–1 shows the loading diagram for the uplift which acts on the section delimited by points a_n and a_{n+1} of the potential slip surface. The ordinates of the loading diagram u_n and u_{n+1} correspond to the vertical distances h_n and h_{n+1} of points a_n and a_{n+1} from points a'_n and a'_{n+1}, respectively. The latter are defined as points of intersection of the equipotential lines ϕ_n and $\phi_{n+}{}'_n$ with the water level ψ_0. By multiplying h_n and h_{n+1} by the unit weight of the water, the magnitudes u_n and u_{n+1} of the hydrostatic pressure (or uplift) at the respective points a_n and a_{n+1} are obtained. The total uplift force U is obtained by multiplying the mean pressure, $1/2\ (u_n + u_{+1})$, by the length $\Delta 1$. If the pore water pressure has been measured u is simply given by its magnitude.

The method presented in the foregoing is preferred since it provides a clear indication of how the uplift affects the resistance of the material along the slip surface. The outcome of measures that would lower the ground-water table or reduce the pore pressure can easily be recognized.

In the finite element analysis of geotechnical bodies, the surfaces of separation are numerous, each element being considered separated by them against its neighbours. Another practical approach for compiling data for the analysis is necessary; for example, a list of nodes is established and the relative magnitudes of the hydrostatic heads according to the positions of the nodal points in the set of equipotential lines are noted. For example in Fig. 7–2, point 175 obtains the value 0.65.

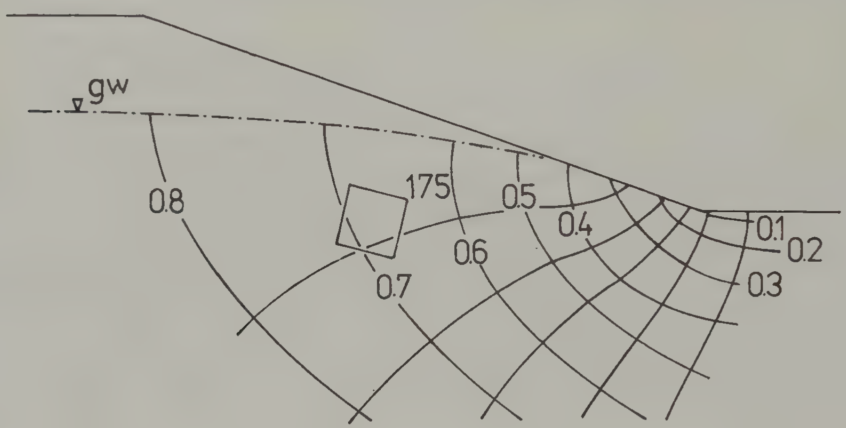


Fig. 7-2. Identification of the position of each nodal point (here e. g. point no. 175) in the field of equipotential lines is necessary as a preparation for finite element static analysis. The numbers attached to the corresponding potential lines (here 0.7 and 0.6) indicate the remaining ground-water head, the head being reduced by seepage (highest level 1.0; lowest level 0). Because of anisotropic and non-homogeneous permeability, the net is usually irregular.

A more difficult problem is that of incorporating the magnitudes of residual forces existing in the mass into the stability analysis. Only approximate corrections can be made if classic stability solutions are used, these being based on experience gained in analyses of other cases. The finite element analysis has proved to be more promising, as will be shown in section 7.6.

The shape and depth of the slip surface also need to be known, especially for classical analysis of the limit equilibrium of slopes. There are several recommended ways of determining the most dangerous potential slip surface, each approach being as good as the method of analysis on which it is based. Some remarks on the position and shape of the slip surface were made in Chapter 4, pointing out, for example, the critical effects of the initial horizontal stresses in the ground, and of the ductile behaviour of the rock. Because these factors were not taken into account in several recommendations, the accuracy of the geometry of the slip surface determined in this way may be low. In the authors' experience it is preferable to guess the positions of two or three potential slip surfaces. The present technique of numerical analysis allows to determine the most dangerous slip surface almost exactly.

The task is simpler when the potential slip surface has been predetermined by geological conditions, or when the static analysis is applied to an existing slope movement. Nevertheless, let it be remembered that the slip surface is substituted by a shear zone where movements of large slopes are concerned.

7.5 Classical solutions

The principle of the equilibrium of a body on an inclined plane forms the basis of numerical analyses of the equilibrium of slopes. The method of considering slices, the most widely applied and highly developed of these methods, divides the slope mass into several bodies, usually vertical slices, and examines their overall stability on the curved slip surface. Individual methods differ with respect to the assumptions made about interactions between the slices along the dividing lines.

The simplest method, Petterson's method, is based on the assumption that the normal stress at the slip surface which gives rise to the resistance to slipping is produced only by the weight of the material placed directly over a given point (Fig. 7–3a).

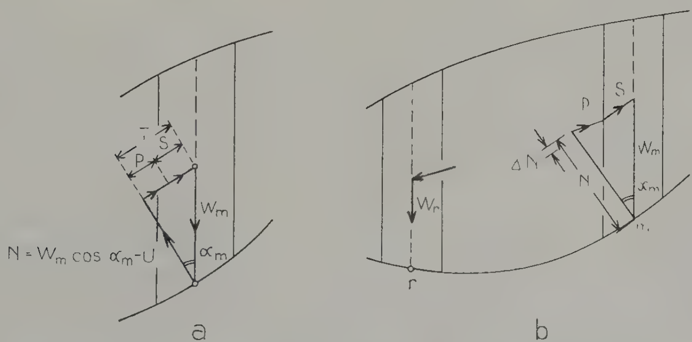


Fig. 7–3. Static interaction between slices, as assumed in Petterson's method of analysis. For the sake of clarity only two slices, m and n, have been considered. The slice weight W is resolved into a normal ($W \cos \alpha$) and a tangential (T) component. The normal component is often reduced by uplift (U). As the resistance (S) against sliding of slice m is not large enough to counter force (T), a force P, which is contributed by the lower slices, is necessary to establish equilibrium. Petterson's analysis (fig. a) is based on the assumption that force (N) is not affected by force (P), i. e. that the direction of force (P) is perpendicular to the direction of the normal force N cos α.

There is no doubt that this assumption does not hold true, since the upper section of the slope is supported by the lower one. Consequently, when it is assumed schematically that slice m is partially supported by slice r (Fig. 7–3b), the force acting between the two slices affects the magnitudes of the forces normal to the slip surface, with respect to each of the slices. It may be inferred from this that the assumptions of Petterson's method do not deviate greatly from reality when the curvature of the slip surface is small. Therefore, application of this method is appropriate in cases of sheet landslides on gentle slopes and in homogeneous soils. The solution yields the factor of safety

$$F = \Sigma[(W\cos\alpha - U)\tan\varphi' + s'\Delta 1]/\Sigma W\sin\alpha$$

(U and Δl were introduced in Section 7.4). As U is subtracted from $W\cos\alpha$, the strength parameters $\tan\varphi'$ and c' reflect the effective stress values.

If the curvature of the potential slip surface is considerable, Petterson's method is not appropriate. Bishop (1955) arrived at two alternative solutions to this problem; the simpler of these considers the effect of the horizontal components E (Fig. 7–4) of the forces P. The vertical components are neglected. Fig. 7–4 shows that the

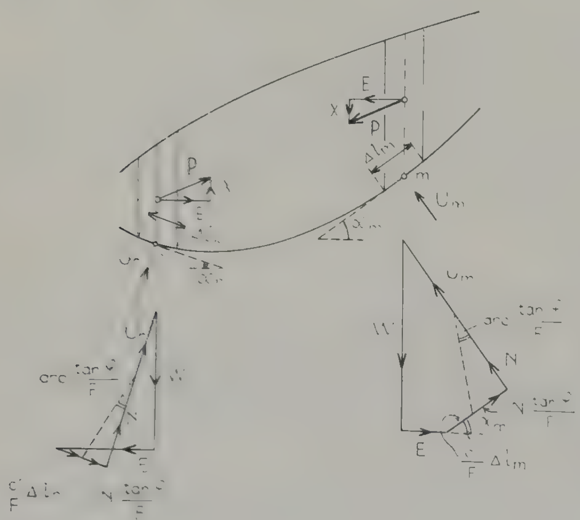


Fig. 7–4. Polygon of forces acting on slices, as assumed in the simpler of Bishop's analyses. For the sake of clarity only two slices are considered.

component E augments the resistance at the base of the upper slices (in this case the slice m) by increasing the N forces. However, the force N acting at the lower slice (r) is increased by forces E when the slip surface is inclined towards the slope, but is otherwise reduced. In this way the stability of the upper slices is found to be increased and that of the lower slices decreased, compared with the result of Petterson's method, and the outcome is closer to reality, as shown in Fig. 4–1c. The changes in the stability of individual slices caused by the E forces correspond with the results of finite element analysis as shown in Fig. 7–5. The line H indicates the imaginary surface of the slope which would be necessary (if Petterson's method would be applied) to produce the realistic normal stresses (line N) on the assumed slip surface. Compared with the actual slope surface, the N forces are relatively greater in the upper part of the slip surface, decreasing downwards, then increasing again along the ascending section of the slip surface near the toe. The analysis yields the formula:

$$F = \frac{1}{\Sigma W \sin\alpha} \Sigma \frac{(W - U\cos\alpha)\tan\varphi' + c'\,\Delta l\cos\alpha}{\cos\alpha + \sin\alpha\dfrac{\tan\varphi'}{F}} .$$

As F is implicitly contained in the expression on the right an approximate solution is necessary, and the resulting magnitude of F is than inserted into the formula.

Fig. 7–5. Results of the finite element analysis of the slope referred to in Fig. 4.1c. The dashed line (N) indicates by its σ-ordinates the computed magnitudes of the normal stress (indicated in MPa) acting on the potential slip surface (p). The dotted line (H) indicates the imaginary slope surface necessary to obtain the above σ-ordinates if Petterson's method was used. Line T indicates the magnitude of tangential stress along the slip surface determined by finite element analysis.

In the case of steep slopes, especially where a resistant layer is present at the toe, the vertical component of the force P cannot be neglected. Fig. 7–6 shows how the presence of more resistant gravel layer tends to produce greater vertical components of the slice-to-slice forces than those produced by the presence of clay layers. The slope is formed by Neogene deposits of clayey and silty sand with layers of silty clay. Static analysis by the finite element method was performed because the slope crown had to support residential buildings, for which a safety factor greater than 2 was set.

Incorporating the vertical slice-to-slice force X in static analyses based on classical methods is not an easy task. Because of the absence of equilibrium conditions, logical guesses are necessary on such matters as, for example, the directions of the forces P. Figures 4–3 and 7–6 indicate to what extent such guesses are possible.

Attempts to refine the classical methods have been handicapped by the fact that the influences of several important factors governing the behaviour of the entire rock mass under investigation, cannot be embraced by these methods. Among such factors are, the role of the initial horizontal forces present in the ground, the ductile behaviour of the material (Fig. 4 – 11) and the influence of the arching of earthen bodies (Section 4.8). The reduction in the bulk factor of safety and the increased danger of progressive failure that consideration of these factors predicts, are, in the majority

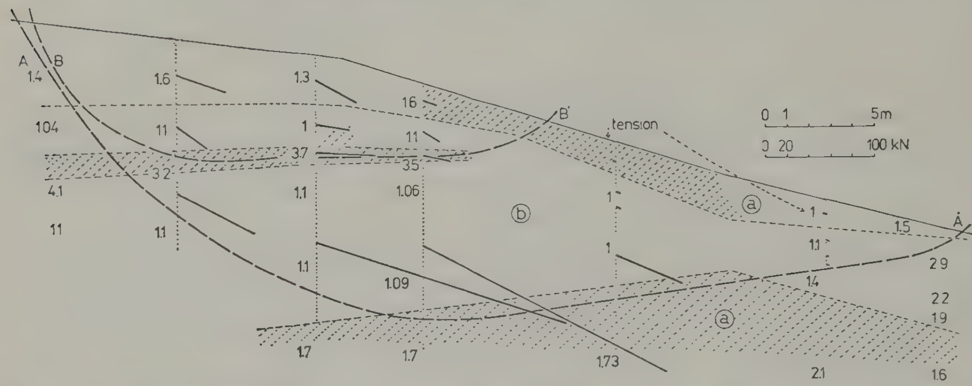


Fig. 7–6. Analysis of the forces (full lines) acting at imaginary vertical boundaries in a slope of the site at Košice. a — Neogene silty clay, plastic to stiff; b — Neogene silty and clayey gravel of medium density. Computed local factors of safety are given. Only a section of a slope is plotted, the valley is three times deeper. The overall factors of safety along the potential slip surfaces *A* and *B* are 4.4 and 3.4, respectively. These magnitudes decrease to 4.1 and 2.9 when the loading by the buildings has been incorporated (Mejzlík and Mencl 1975a).

of cases, of greater practical relevancy than the increases in the factor of safety obtained by improvements to the classical methods. Even in a simple case such as that presented in Fig. 4 – 1(c), the classical method yields a factor of safety of 2.1, while finite element method gives 1.8.

Objections may be voiced on the grounds that in principle the classical methods are limit equilibrium methods, whereas the finite element method – at least as it is applied in the present book – deals with real forces and stresses, etc. Nevertheless, the negligence of the progressive failure development is an important drawback of the classical analyses.

The classical methods of stability analysis are useful when evaluating the influence of the stiffness of the sliding body. They are also helpful as a mean of comparing the effectiveness of different corrective measures. The absolute magnitudes obtained for the safety factor may often be higher than reality, as mentioned in Chapter 4, and thus Skempton (1969) introduced a correction factor (R). Referring to the data given in Chapter 4, the magnitude of R would have to be of the order of 1.17 for a slope 16 m high, gradient 1 : 3, in stiff clay exhibiting a coefficient of horizontal stresses $K_0 = 0.75$. If the value of the latter becomes $K_0 = 1$, R increases to 1.36, and with $K_0 = 1.5$ R approaches infinity, because the slope is not longer stable. It seems probable that when a great deal more experience has been gained, use of the R factor will become an invaluable aid to analysing the stability of slopes.

7.6 The finite element method

This method has widespread application in the static analysis of slopes. The various solutions presented in this volume have been obtained by using the finite element method. Before starting a numerical analysis of this kind, there are some important points to be considered in order to simulate properly the chief physical processes that are going on in the slope. As was already explained in section 6.3.4, the pseudo-elastic approach is postulated in the present volume. The linear elasticity does not yield correct solutions (Mencl 1977). Therefore the stress dependence of the stress-strain parameters is to be introduced (Section 6.3.4). The mechanical effects of dilatancy are not negligible in many cases and therefore one of the important questions is how to introduce the volume increase of dilatancy when the finite element method is applied. Several theoretical assumptions in the theory of plasticity cannot be applied in this respect because:

The volume increase occurs within the thickness of the slip surface only, and not throughout the entire strained rock or soil mass.

The volume increase does not correspond with the total angle of shear resistance of the material – only with that component of it induced by dilatancy.

The process of strain-dependent volume changes begins when the factor of safety begins to decrease below the magnitude of about 1.6.

With increasing compressive stressing the magnitude of the dilatancy decreases (line D in Fig. 7–7) so that a volume decrease (contractancy) begins to appear.

Moreover, the matrix calculus forming the basis of the finite element method does not allow for values of the Poisson ratio higher than 0.5 (in practice, 0.48), in which way the increase in volume could otherwise be simulated.

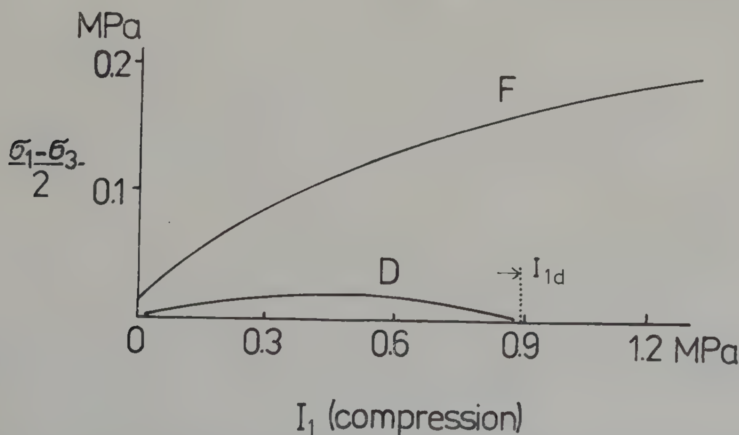


Fig. 7–7. Contribution of dilatancy (D) to the shear strength (F) of a Neogene stiff clay (Králová, Slovakia; depth 24 m, $w = 24\%$, n = 39%, when $I_1 = 0.25$ MPa in compression, $w_p = 23\%$, $w_L = 54\%$). Computed from the results of triaxial tests of Havlíček, 1974.

For those reasons the solutions reported in this volume have been obtained using the following technique (Mejzlík and Mencl, 1973):

(a) The first step in bringing dilatancy to bear on the problem is to use the postulates of the classical theory of plasticity.

(b) The results obtained are corrected in the next iterative step of the redistribution of forces in order to obtain the real magnitudes of the dilatancy. For this purpose, four parameters are introduced: s – the thickness of the shear surface, D – the increase in s owing to dilatancy (expressed as a ratio), a value which is strain-dependent, B – the shear strain together with which the dilatancy arises (a value which is dependent on the intensity of compression stresses), and I_{1d} – the first invariant of stresses (the sum $\sigma_1 + \sigma_2 + \sigma_3$) at which dilatancy turns into contractancy. For example, in cases involving stiff clay (which appears in many of the examples quoted in the work), the following values were applied: $s = 0.001$ m, $I_{1d} = 0.9$ MPa, $D = 0.3$ (decreasing to zero at $I_1 = I_{1d}$), $B = 0.5\%$ when $I_1 = 0$ (increasing to 3% at $I_1 = I_{1d}$).

(c) The increase in volume caused by dilatancy affects only the thickness of the slip surface (s), as already noted above, and therefore the magnitudes of D must be re-evaluated in relation to the linear dimension of each individual element. This part of the procedure is incorporated into the computer program.

The overall factor of safety of the slope can be obtained on the basis of the normal and shear stresses resulting from the computation. This procedure can also form a part of the FEM-program and can be carried out along any assumed slip surface. An important indicator, namely the value of F, deserves attention here: when during the loading steps of the computation any element enters the state of $F < 1$, a redistribution of forces is necessary in order to unload the given element and to restore its F value to a number near to unity. The greater the number of elements affected in this way, the slower is the process of redistribution of forces, and thus the number of iterative steps taken by the computer can be taken as some indication of the safety of the slope. When repeating the calculation with some of the reduced strength parameters, a very reliable picture of the stability of the slope can be obtained.

7.7 The deformation approach to the stability of slopes

An important question arises with respect to the stability obtained by the finite element technique: because the method is more exact than the classical methods, can the magnitude of the safety factor required be reduced? As already indicated, it is helpful to take account of the percentage of elements displaying $F = 1$, or to evaluate the iterative process of the redistribution of forces. However, another criterion seems to be promising, namely the deformation behaviour shown by the finite element method.

In Figs. 7–8 to 7–10 three different slopes are shown. The first two belong to a group of 45 profiles analysed for the satellite town at Košice (see also Fig. 7–6). The computed vectors of displacements produced by the weights of the proposed buildings are included. These two slopes differ in stability; whereas the safety factor of the first is of the order of 1.25, that of the second is about 2. And this is reflected in the directions of the vectors of displacements, the tendency for displacement

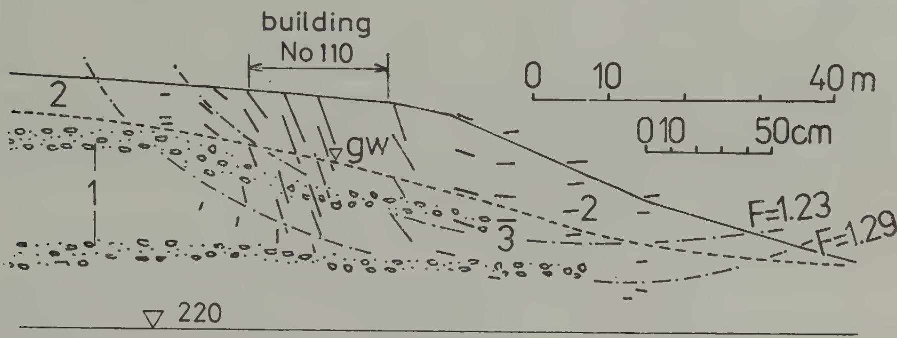


Fig. 7–8. Profile 2 of a slope at Košice (Carpathian Neogene basin); 1 — silty sand with gravel, locally disturbed by ancient landslides, 2 — stiff silty clay, 3 — weak silty clay. Overall factors of safety along two assumed slip surfaces are shown. Computed vectors of displacements produced by the weight of the proposed building are included. As expected, building 110 cannot safely be constructed (Mejzlík and Mencl 1975a).

towards the slope face being quite evident in the lower stability situation. From an evaluation of the results of 17 analyses, it seems that the tendency for movement towards the slope face appears when the factor of safety is less than about 2.1 when clayey soils are concerned. This is also a reasonable safety factor for the construction

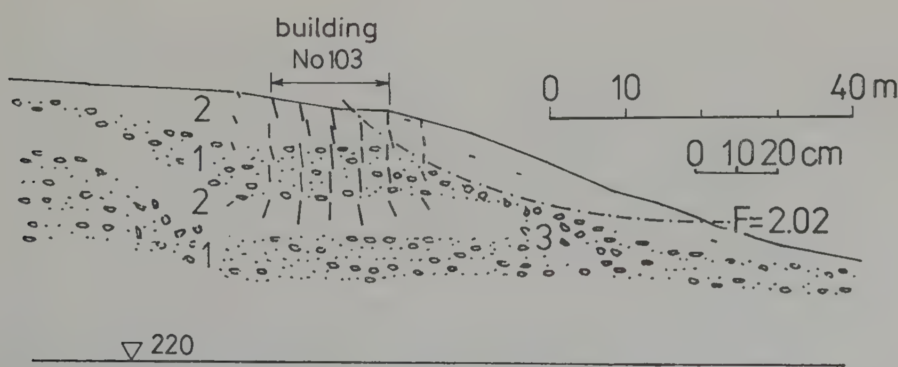


Fig. 7–9. Profile 5 of the slope referred to in Fig. 7–8. In contrast with the latter the safety factor for the slope (including the weight of the building) is about 2, and this is manifested by the directions of the displacement vectors, produced by the weight of the building; 3 — disturbed sands.

of buildings at the top of the slope. Where sand is concerned this threshold seems to be much lower, that is about 1.5 – a striking coincidence with the traditional approach, and the figure is much lower still when we consider rock slopes; Fig. 7–10 shows the tendency towards steeply dipping displacements in a rock slope. The overall

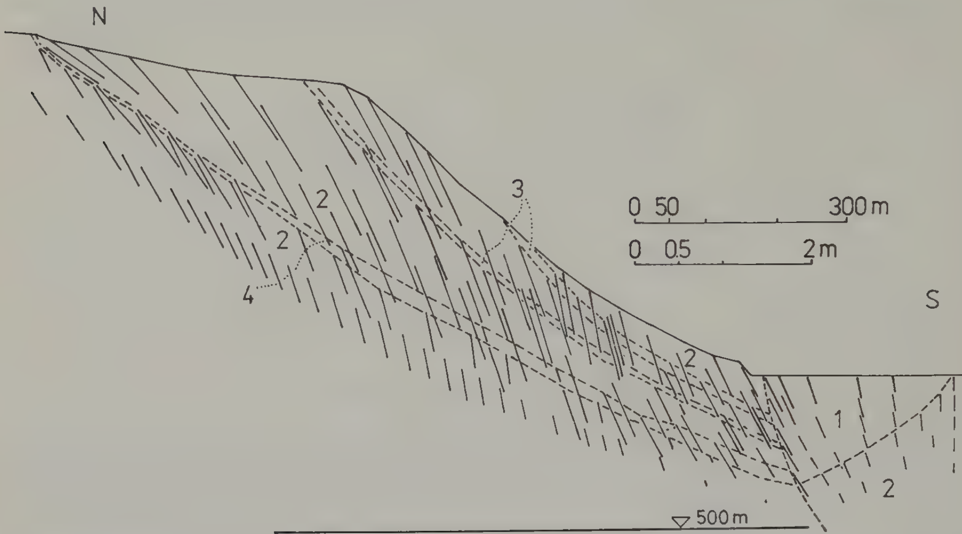


Fig. 7–10. Northern slope of the valley on the power-plant site of Černý Váh, Slovakia, in the overthrust structures of the Central Carpathians; 1 – tectonic breccia (Lower Triassic), 2 – dolomite without parallel orientation (Middle Triassic), 3 – zones representing joints extending less than 30 m and spaced more than 8 m apart, in the dolomite, 4 – main fault zone. The purpose of this figure is to show the calculated vectors of displacements after adding 5% to the unit weight of the rocks. For the sake of clarity the displacements of all the nodal points were not drawn (Mejzlík and Mencl 1978).

safety factor of the latter slope is 1.34 along fault 4. The displacements were calculated for a 5% addition to the unit weight of the rocks.

In order that the static behaviour of a slope can be followed after some small displacement has occurred, it is necessary to introduce new coordinates for all nodal points after each step of loading (the so-called 2nd order analysis in mechanics).

7.8 Physical models

The necessity of making a preliminary analysis where large displacements are involved arises very much in connection with underground mining. Physical models are capable of satisfying this requirement. In order to approximate the real situation as closely as possible, the technique of equivalent material modelling has been introduced.

With regard to slope stability the more advanced phases of displacement are generally less important than the onset of the failure itself, and therefore mathematical solutions have usually been sought. There are, however, several exceptions. For example, the development of progressive failure where arching of the rocks is expected (Section 4.8) can be reproduced qualitatively by a physical model and then analysed quantitatively by simple numerical methods. For these and similar purposes, several new techniques and methods of analysis have been devised. Gulakyan (1977) described an application of the equivalent material technique in which local heating of a part of the model was tried in order to obtain viscous behaviour of a fatty modelling material as a simulation of the creep of clayey rocks. Kohoutek (1979) argued that physical models are indispensable, even where deformations of a small degree are considered.

7.9 The "cohesion approach"

As already discussed in sections 6.3.4 and 6.3.5., damage to the structure of earthen materials exhibiting cohesion occurs long before their strength has been overcome under mechanical loading. Therefore, the question arises as to whether the design of slopes should not pay attention to protection of the structural bonds before damage occurs. Naturally, several conditions and limitations which are inherent in such an approach must be considered:

Earthen masses, in which sliding displacement can develop along continuous separation planes cannot be involved.

When large intrinsic stresses are present in the ground (sections 4.3 and 7.10), damage to the structure of the material begins to occur under conditions associated with a relatively high overall factor of safety, and therefore situations of this nature must be excluded too.

Significantly non-homogeneous earthen masses undergo a typical progressive failure and the stress concentrations in the brittle "dowels" cannot be analysed in a simple manner.

In spite of these limitations, a wide range of cases still remains to be discussed. As an example, let us take the case of the slope illustrated in Figs. 4–6 and 4–7 (section 4.2). The slope is assumed to have been formed by excavation in stiff fissured clay of which the intrinsic horizontal stresses were defined as $K_0 = 1$, and the strength parameters are given by $c' = 0.025$ MPa and $\varphi' = 18°$. The slope is 20 m high and inclined at 18.5°. Its "classical" general factor of safety is about 2.1, or that one obtained by the finite elements method, about 1.6.

Two limitations should be placed on the shear stressing of rocks and soils in order to prevent inception of damage of the internal structure:

(a) The limitation indicated by the onset of dilatancy. Line D in Fig. 6–27 corresponds to this requirement; $c'_d = 0.015$ MPa and $\varphi'_d = 14°$ for the specified clay.

(b) The limitation indicated by the inception of contractancy.

The latter limitation does not apply to the case we are considering here (see also Fig. 4–11). The former limitation yields a safety factor of about 1.4, if Bishop and Morgenstern's diagrams are used. The margin of 0.4 seems to correspond to the "progressive failure" difference between the above values of 2.1 and 1.6 for the factor of safety.

In the author's experience, the limitation (b) is of considerable importance for the slopes of deep open pit mines. The problem of designing a slope in an open pit mine in a Tertiary basin in North Bohemia serves as an example (Mencl and Seyček 1976). The slope height (which includes claystone beds overlying the coal seam) is 82 m and the ground-water has been drained by two rows of pumping wells reaching down to the Cretaceous bedrock. Typical results of one of several series of drained, 10 cm diameter triaxial tests of the clayey material (depth 50 m, $w = 23\%$, $w_p = 40\%$, $w_L = 75\%$, $S_r = 83\%$) are presented in Figs. 7–11 and 7–12. Lines C and D are

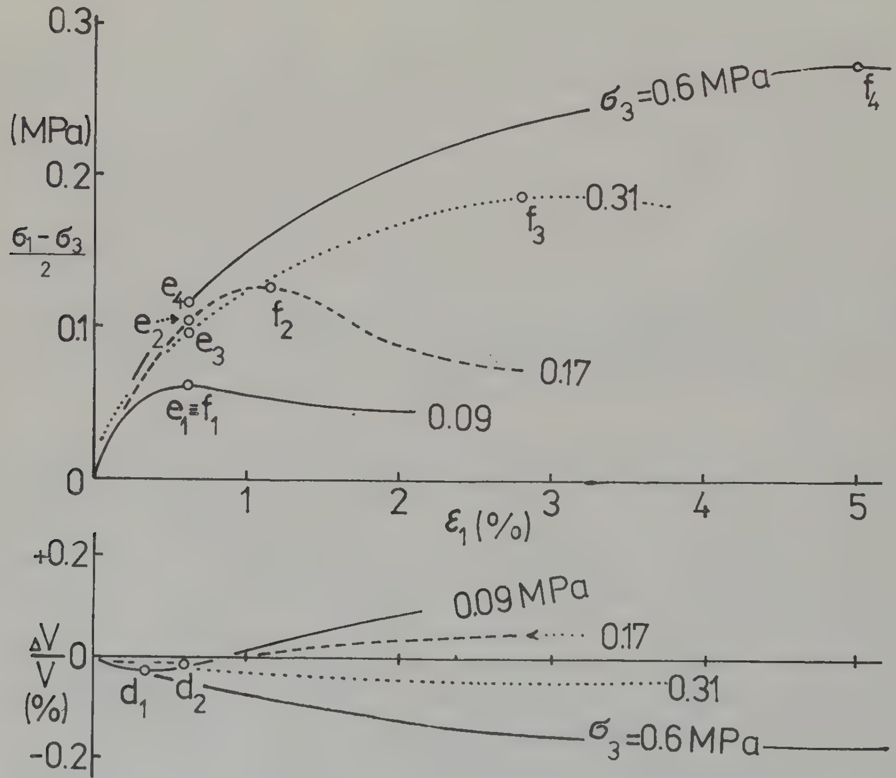


Fig. 7–11. Stress-strain diagrams resulting from drained triaxial shear tests on clay overlying brown coal in an open-pit mine near Chomutov, Bohemia. The specimens were 10 cm in diameter; f — shear strength limits, e — limit points for the development of equal volume changes, d — inception of dilatancy (Mencl and Seyček 1976).

plotted according to the diagram in Fig. 6–27. Because deep ductile yielding of the clay may develop along the possible slip surface, line C must be taken into account in the analysis. Nevertheless, when it is considered that both brittle (dilatant) and ductile (contractant) modes of failure may develop according to the depth of the potential slip surface, a third limitation to the shear stressing of the material has to be imposed. This requires that the shear stressing does not generate large differences in the volume changes of the tested specimens, whatever the magnitudes of σ_3. This limitation (indicated by points e in Figs. 7–11 and 7–12) gives a value for the shear resistance of about 16°. On the basis of the latter a slope gradient of 13° is arrived at under the provision that the ground-water is to be kept below the level of the head of the coal seam.

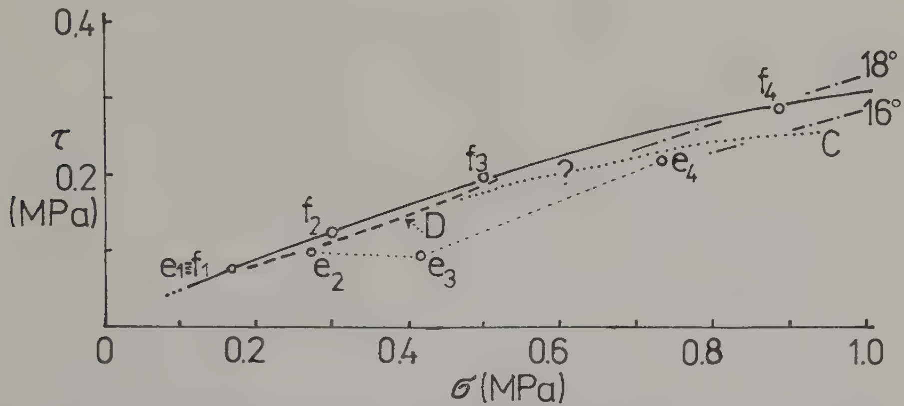


Fig. 7-12. Mohr diagram for the results of the tests referred to in Fig. 7-11; $f_1 f_2 f_3$ — shear strength envelope, D — line representing points d in Fig. 7-11, C — approximate limit of the inception of contractant behaviour; e, and f — as in Fig. 7-11.

The condition of the northern slope of the mine justified the outcome of the analysis. This 15-year-old slope is 95 m above the coal seam, dipping 10° in the first 20 m below the ground surface, where Quaternary deposits were excavated. They have suffered from slow sliding movements because they have been infiltrated by water escaping from a reservoir, the earth dam of which is situated only about 50 m away from the slope crest. The remaining 75 m in the Neogene clayey soils dips at an angle of 14.5°, and therefore the general angle of the slope is 12.8°. The Neogene deposits have been drained by the galleries of the underground mine which adjoins the open pit mine.

7.10 The "horizontal equilibrium approach"

As already discussed in section 7.5, classical computations of factors of safety of slopes are in fact ultimate load state analyses (Záruba and Mencl, 1976, sect. 3.7). Therefore they do not consider the existence of the horizontal compressive forces which result from the initial horizontal stress originally present in the ground. These forces are supposed to be small when the imaginary limit state of stability is conside-

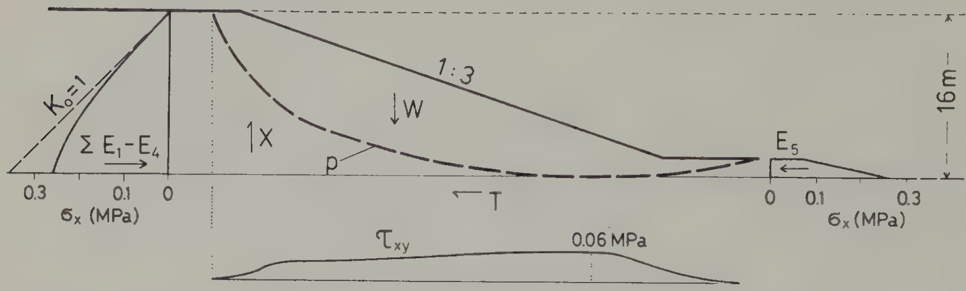


Fig. 7–13. Slope of a cutting in stiff plastic clay, $K_0 = 1$, no ground-water. Loading diagrams of horizontal forces $E_1 - E_4$, T and forces X acting on a body of soil located above the potential slip surface p.

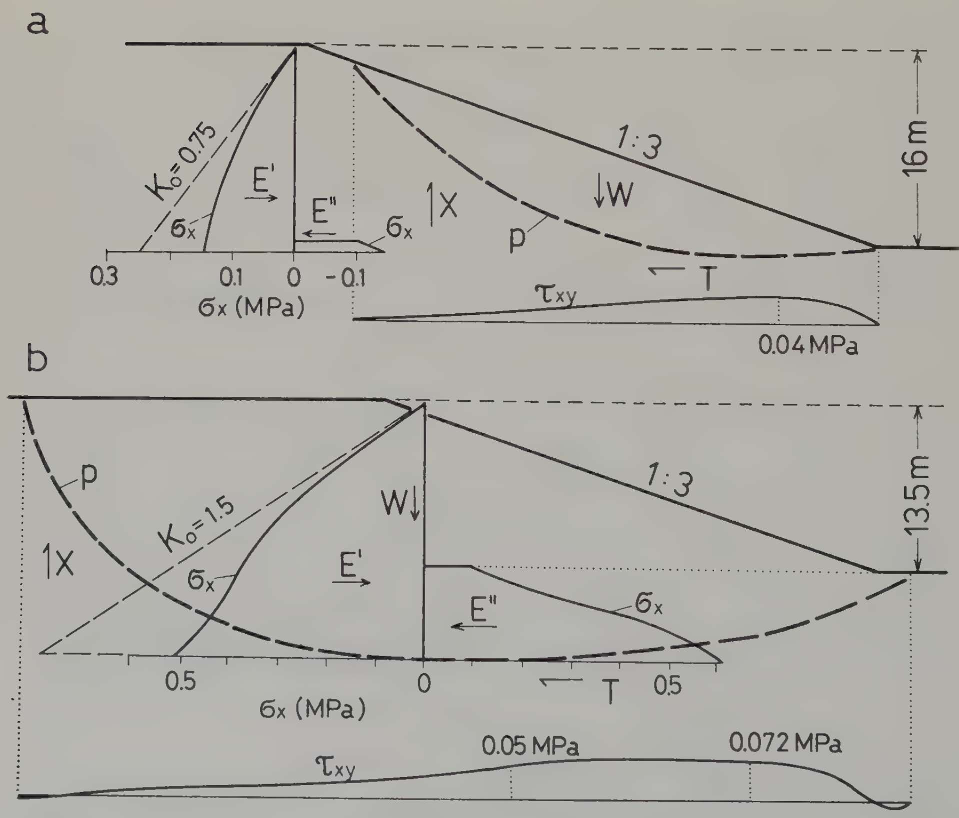


Fig. 7–14. Forces as referred to in Fig. 7—13, but for (a) $K_0 = 0.75$, $D = 16$ m, (b) $K_0 = 1.5$, $D = 13.5$ m, no ground-water.

red. But they are present in the slope in its actual state. The magnitudes of these forces have been indicated in Fig. 4–7(b) for a slope of a 16 m deep cutting in a stiff clay with $K_0 = 1$. The compression forces tend to translate the slope to the right and are resisted by shear forces T_1 to T_4, as well as by the compression force P_5. The sum of P_1 to P_5 forces is 2384 kN. In order to obtain a factor of safety, this sum is to be compared with the sum of the horizontal components of the shear strength the soil is capable of mobilizing along the potential slip surface. This resistance is given by (Fig. 4–7b) $S = (W - \Sigma X) \tan 18° + Lc = (6700 - 374) \tan 18° + 52 \times 25 = 3355$ kN, which yields the factor of safety $F = 3355/2384 = 1.41$, a value not very different from 1.6 found by the finite element analysis (Section 4.2).

Let us observe that:

(a) The diagram of the horizontal shear stress along the potential slip surface (Fig. 7–13) is rather uniform without stress concentrations, so that the analysis as given is admissible. (b) Because force ΣX is relatively small, the sum of E_1 to E_5 forces is the only important value which is unknown. The diagrams plotted on the sides of Fig. 7–13 show that the existing horizontal stresses can be obtained by somewhat reducing the diagrams of the initial horizontal stresses. (c) In order to make the solutions accessible the authors intend to elaborate a series of charts showing the reduction which is to be made in typical geological situations of slopes.

Two solutions are presented in Fig. 7–14. For the case shown in Fig. 7–14(a) the resulting „horizontal" factor of safety is 1.94, which is to be compared with the value of 1.8, obtained by the finite element analysis. For the slope shown in Fig. 7–14(b) the "horizontal forces" analysis yields $F = 1.6$ and the finite element analysis $F = 1.4$.

Chapter 8

CORRECTIVE MEASURES

8.1 Scheduling of the stabilization work

The stabilization of landslides or slopes prone to sliding movements must be executed according to a well thought-out plan, which lists individual measures in order of urgency. It would be wrong, for instance, to begin the filling of a stabilization berm of an embankment before draining the subsoil. On the other hand, a long and often laborious process of investigation, static analysis, and planning of the stabilization work should not delay the measures that obviously have to be taken; the treatment of active landslides is always a contest with time. The first remedial measures should include:

Capture and drainage of surface water flowing into the slide area or emerging in the head scarp area.

Pumping of water from all wells in the slide area and from any undrained depressions.

Filling and compacting of the fill in all open cracks which could be entered by surface water. This particularly concerns deep cracks which develop during slope movement and which reach down to the slide plane.

In the case of slow, creep-like slope movements, the packing of open cracks may also have an impeding effect, because it can hinder upslope propagation of slope deformation.

If it is geologically feasible, one or more trial horizontal drainage boreholes should be drilled. With the exception of cases in which a lot of damage is threatened by the landslide, systematic subsurface drainage should wait until after the investigation work.

The scheduling of corrective measures must pay due regard to weather conditions. In Central Europe extensive operations are very difficult or even impossible in winter, when the surfaces of waterlogged slide areas may not be accessible. Therefore, the individual operations should be scheduled with caution so that the remedial works would not remain unfinished. In such a case it is better to postpone the operations to spring time.

All corrective installations must be regularly checked and maintained. If regular maintenance is not carried out, or if the agreed programme for developing and using the slide area is not adhered to, then extensive and costly corrective measures may come to nought, and within a short period new movements may start up. The schedule of inspection and the maintenace work should be included in the overall planning of the corrective measures.

8.2 Treatment of slope conformation

The stability of a slope may be increased either by reducing the volume at the head or by expanding the volume at the toe. The former treatment is particularly effective and an example is shown in Fig. 8–1.

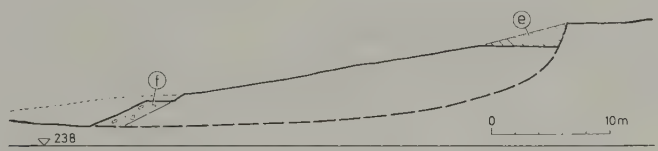


Fig. 8–1. Failure of a slope caused by the excavation for a road in a stiff Neogene clay in Brno. A small excavation *e* increased the factor of safety of the slope from 1 to 1.16, although the excavated volume was only 4.5% of the volume of the sliding mass; $\varphi_r' = 11.5°$, f — protective fill.

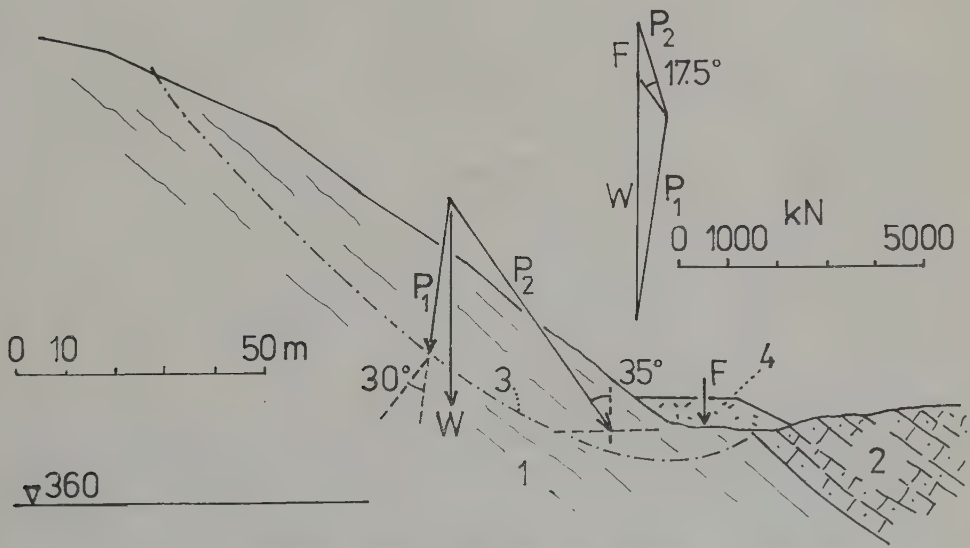


Fig. 8–2. Stabilization of slope movement in the Cretaceous marlstone (1), Štramberk, Moravia. The assumed shear strength angle was 30° along the bedding planes and 35° across them in the assumed slip zone (3). From these values, components P_1 and P_2 of the weight W were estimated. The road embankment constituted the auxiliary weight F needed to produce a resultant with force P_2 acting at an angle of 35°/2, in order to attain the stability of the slope. 2 — Cretaceous sandstone.

The presence of a lower layer possessing a large shear strength angle makes building up of the toe more advantageous because a small extra weight may be sufficient to stabilize the slope foot (Fig. 8–2). On the other hand if the strength of the bottom layer is low (e. g. in coal beds or in unconsolidated clay layers), it is also possible to place a fill in front of the slope, as is frequently done in open pit coal mines (section 13.2.4). An important factor which gives a good buttressing effect from the fill is proper drainage of its base, particularly if the fill rests on clayey material. The best drainage can be accomplished with a gravel layer 0.6 to 1 m thick. This procedure is mostly adopted in road engineering, but in coal mines, where access to gravel may be difficult and the relative size of the buttress can be greater than in road engineering, drainage trenches are generally regarded as sufficient. There is a tendency to neglect drainage in some mines, particularly in the loam pits of brick factories.

The use of the drained fill also works well in the toe areas of moving slopes, where saturation of soils is the chief problem. Generally, there is insufficient time for installing horizontal drainage boreholes, whereas the fill can be dumped in a very

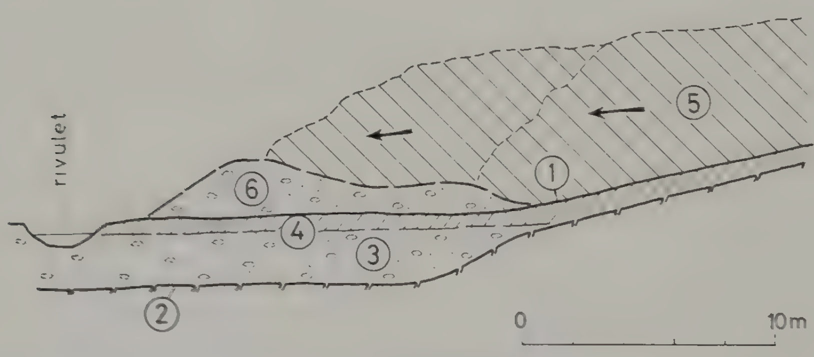


Fig. 8–3. Schematic cross-section of a drainage and buttress fill dumped in front of the face of a sliding mass; 1 — original ground surface, 2 — bedrock, 3 — gravel terrace, 4 — surface soil, 5 — toe of the landslide, 6 — drainage fill.

short time, even overnight. Owing to the velocity of the movement, the gravel fill (Fig. 8–3) may need to be placed some distance from the current position of the toe of the slide. The overthrusting material at the front of the sliding mass becomes drained and cessation of the movement follows. A successive treatment of the sliding mass with horizontal boreholes can therefore be postponed.

A protective fill dumped on the surface of slopes excavated in clays is another effective measure, if carried out in good time. Fig. 8–4 illustrates the problem. The slope is the same as in Fig. 4–1(c), but the coefficient of initial lateral stress was assumed to be unity and the presence of ground-water was assumed. The resulting stress state, illustrated by points (3) is considerably safer than that of points (2).

Therefore, the sooner the protective fill is put in place, the greater is the safety of the slope. This is a point that needs to be emphasized in engineering-geological reports.

The design of the protective gravel fill should take account of the foregoing considerations, and therefore the fill should be designed as a self-supporting berm, e.g.

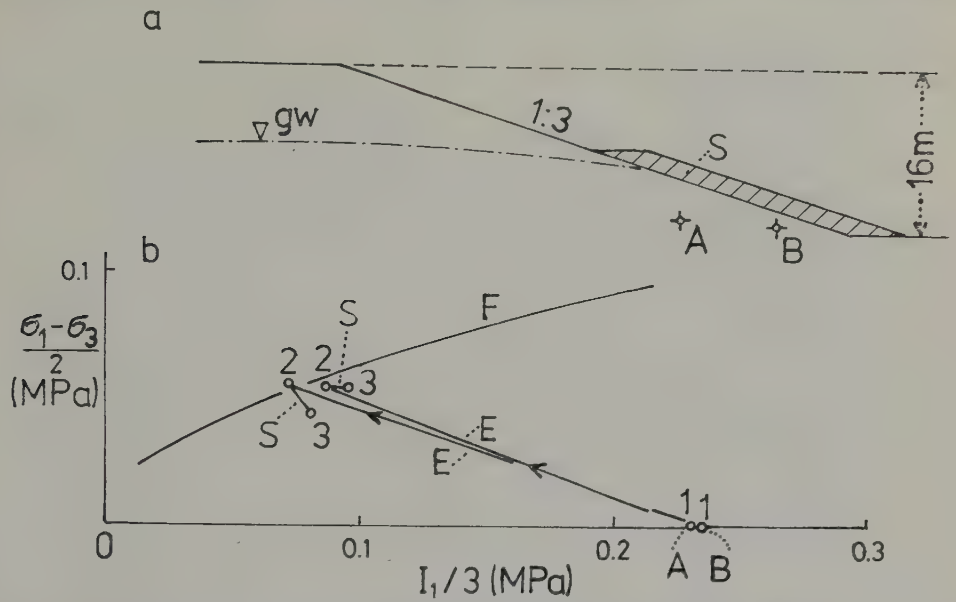


Fig. 8-4. Protective fill (S) built on 1 : 3 slope of a cutting 16 m deep in Neogene clay. The fill alleviates the stress in the slope, as illustrated by the stress states at points (A) and (B). The diagram shows the initial stress states (points 1), the states obtained after excavation (E) (points 2), and the states generated by the protective fill (points 3). The intensities of the mean compression stress and shear stress form the coordinates of the diagram.

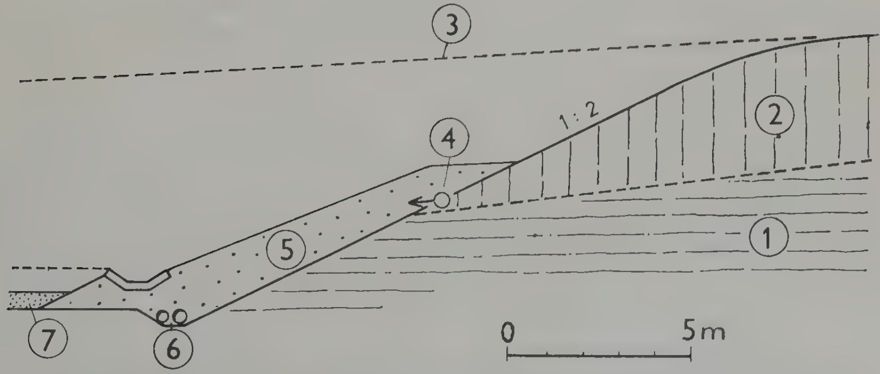


Fig. 8-5. Arrangement of a supporting fill in a cutting; 1 — Neogene clay, 2 — loess loam, 3 — original ground surface, 4 — water seepage, 5 — gravel fill, 6 — drainage, 7 — consolidated subgrade used as a service road.

it should not bear against the structure of the roadway shown in Fig. 8–5. There is no need to emphasize that only the *inside layer* of the fill requires to be made of the permeable material.

Thicker protective fills work well where the banks of reservoirs are concerned. Fig. 8–6 shows a profile of the northern bank of the Liptovská Mara reservoir, in the Flysch basin of the Western Carpathians.

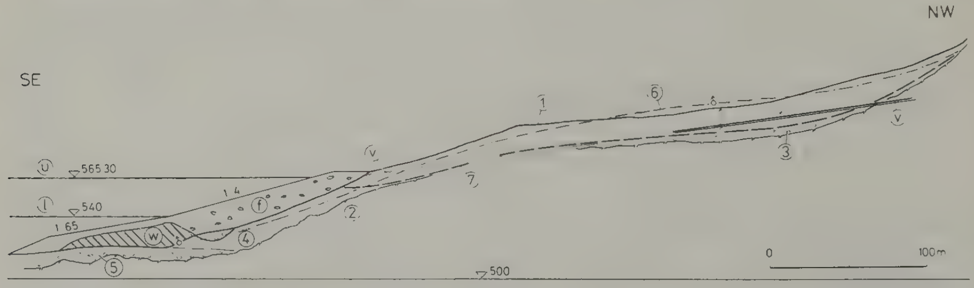


Fig. 8–6. Profile of the northern bank of the Liptovská Mara reservoir; 1 — original ground surface, 2 — bedrock (Palaeogene), 3 — rock fragments at the base of the colluvial material, 4 — clayey debris of the landslide, 5 — terrace gravel, 6 — original ground-water table, 7 — slip surface, f — protective gravel fill, w — waste fill, 1 and u — lower and upper operating levels of the reservoir, v — drainage boreholes.

8.3 The drainage of landslides

8.3.1 Surface drainage

The surface of any area affected by sliding is generally uneven, hummocky, and traversed by deep fissures. In the depressions and fissures water accumulates, and wet grounds develop. Therefore one of the first remedial measures is the surface drainage of the slide areas. Although surface drainage by itself is seldom sufficient to stabilize a slope which is in motion, it can contribute substantially to the drying out, and thus also to the control of the landslide.

First of all, all streams and temporary watercourses are diverted from the threatened area. In addition, all springs issuing within the slide area, especially those at its head, must be contained and diverted away from the slide. For an immediate, provisional diversion of flowing water any available pipes may be used. Surface waters on the Handlová landslide were led away through air pipes lent by the Handlová Mine, and fire-hoses. In the first stages of a landslide when the movement and changes in the relief are appreciable, surface pipes have the advantage of being easy to move and they are inexpensive. In winter, however, they prove less suitable because they do not protect the drainage water from freezing.

After partial stabilization of the landslide, open ditches of adequate dimensions and gradient are excavated for discharging rain-water. At the same time, the ground surface is levelled and undrained depressions filled along with all cracks so that a continuous run-off of surface water is ensured. During these operations the grass cover must not be disturbed unnecessarily, since grass reduces the tendency of water to percolate down into the slope (Fig 8–7).

Fig. 8–7. Handlová landslide; drainage ditches paved with concrete tiles (photograph by Záruba).

The arrangement of ditches depends on the soil type; their banks and floors must be sufficiently firm so as to resist erosion. They are paved either with natural stone of suitable properties, or with concrete tiles set in a sand foundation, the joints being sealed with cement or sod. Water infiltrating into the sand bed is directed along the ditch by establishing low steps. In sandy soils, the ditch sides and bottom may be consolidated with asphalt, bitumen or oil sprinkle.

In some cases ditch tiles of reinforced concrete have proved suitable for surface drainage. These tiles are slightly narrowed at one end so that they can be inserted into one another. Compared with paving, gutters of reinforced concrete have the advantage of being less pervious; they can withstand slight movements of the slope. Wooden troughs, which are occasionally used, can easily be set out and then repositioned if necessary, but with frequent moistening and drying out, the wood deteriorates and becomes more leaky.

In addition to ditches constructed in the slide area, peripheral ditches above the head scarp are sometimes dug so as to divert surface water flowing down adjacent slopes into the potentially unstable area. They must be provided with impervious paving and have a uniform gradient to prevent deposition of material on the bottom of the ditch. Water retained in blocked ditches can cause additional disturbance of the slope, and may seriously upset the stability, even in the case of a slide which is temporarily at rest (Fig. 8–8).

Fig. 8–8. Handlová landslide; continuous maintenance of the paving is necessary, as even small movements disturb the ditch and impair its function (photograph by Záruba).

8.3.2 Subsurface drainage

In Chapter 4 the role of uplift, and forces generated by ground-water flow in the origin of landslides was discussed. As ground-water is one of the major causes of slope instability, subsurface drainage is a very effective form of remedial treatment. It complements or may even render unnecessary the shape adjustment of slopes, since a drained slope may be stable at a steeper angle than an undrained one. The disadvantage of subsurface drainage is that the drainage system cannot be designed until after geological and hydrogeological research has been completed, with the result that it comes into operation somewhat belatedly.

Vertical exploration borings converted into pumping wells are free of this disadvantage. The diameter of the borings, however, must be larger than that needed for exploration alone, since the diameter of the casing for the pumps requires a diameter of at least 219 mm, and even if the width of the filter is minimal, the diameter must then be 280—300 mm. The progress of boring is thus slower and investigation of the area consequently delayed. On the other hand each completed boring represents an effective pumping well.

If old water wells exist at the locality, they must immediately be pumped dry, the local inhabitants may be persuaded to help empty the wells in their own interest. Where the water rises close to the surface, the pumps of the fire brigade may prove convenient, as they are easily transportable, highly effective, and are powered by petrol or diesel engines. As the network of exploration and pumping holes increases in density, a picture of geological and hydrogeological nature of the area emerges with enough precision to form the basis of definitive drainage measures.

Drainage galleries are conventional deeply situated structures such as were built in the first railway construction schemes of the last century. They have several advantages, first of all serving as a means of investigating water percolation through the rock, and thus helping to ascertain precisely the hydrogeological conditions of the slope. They are capable of discharging a large amount of water. Their effectiveness may be increased by making long or short drainage borings in the walls, floor or roof of the gallery. Thus, galleries can be constructed below the slide plane for the purpose of collecting water from the overlying layers through vertical boreholes. If the water seems to derive from the more permeable bed in the floor of the gallery, a shaft or a trench may be dug. The course of the gallery may be changed so as to follow the influx of water, or to make contact with the lower ends of vertical drainage boreholes. The diameter of galleries is generally so large that they discharge water even when partly disturbed.

Yet, drainage galleries have several disadvantages, too: firstly, they are costly, since the drainage of a few litres of water per second requires a large engineering operation. The effectiveness of drainage galleries is sometimes unjustifiably criticized by laymen who are concerned that an "underground lake" may be tapped. The driving of galleries into already disturbed, slipped rocks is laborious and made hazardous by the possibility of caving in. The use of mechanical vehicles nowadays for loading and transport undoubtedly lowers the digging costs, but necessitates increasing the dimensions of the galleries and thus raises the cost of filling. Drainage galleries should not remain empty, and are usually backfilled with stone or gravel to ensure that a good drainage capacity is maintained even if partial deformation occurs. The backfilling tends to be an expensive, labour-intensive operation.

In spite of these disadvantages, galleries still represent an indispensable method of draining deep slides in which the distances to be drained are greater than 250 m — the maximum length yet attained for drainage borings in soft rocks. The use of these galleries in the stabilization of the Handlová landslide is shown in Fig. 5—13, and an example of successful drainage of a canal slope is shown in Fig. 5—36; boreholes were drilled inside the gallery in order to drain saturated pockets of terrace gravel.

From our experience drainage boreholes cost no more than a fifth of that of drainage galleries. The cost is largely reduced on account of the shorter construction time which depends upon suitable machinery being available, and the almost no need for pumping costs compared with pumping wells. There are four drawbacks involved in drainage borings: (a) It is difficult to guarantee that a borehole will contact those water-bearing beds in which the pressure of the ground-water is responsible for impairing the stability of the slope. (b) Even if an appropriate layer is contacted, the reduction achieved in the uplift force may not be sufficient to increase the shear resistance. (c) The maximum length of effective borings is 250 to 300 m, but at this length the position of the end of the borehole may deviate from the intended position by several metres. Fine sands or sandy gravels often do not permit the drilling of boreholes longer than 100 m, because the fine grains jam the drilling tool. (d) Drainage boreholes have a limited useful life.

As far as items (a), (b) and (c) are concerned, the following observations may be made: Unless the most permeable layer within the slope has been reached, the drainage process will not be fully effective. Effective drainage by attacking the less permeable talus loams requires several times the number of boreholes that would otherwise be needed for draining the underlying rock debris or permeable bedrock. To illustrate this, two specific cases may be compared.

The slope movement at Příluky (Fig. 8—9) involved a thick mass of colluvial detritus on a slope consisting of Palaeogene rocks. Although the width of the endan-

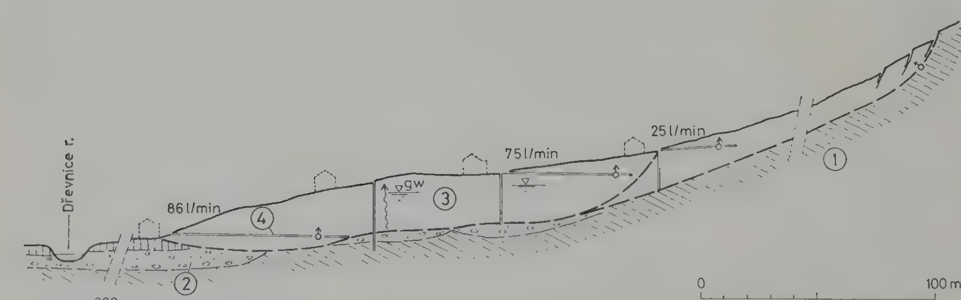


Fig. 8–9. Cross-section of the landslide at Příluky near Gottwaldov (Moravia). The total movement occurring in 1961 was 1.5 m and destroyed three houses. The discharge of three horizontal drainage boreholes decreased from 169 litres per minute (in 1961) to 105 litres per minute in 1978; 1 — Palaeogene sandstones and claystones, 2 — terrace gravel, 3 — slipped loam and detritus, 4 — drainage borings.

gered area was 150 m and its length more than 300 m, seven drainage boreholes were sufficient to stabilize the slope. Of these, only three reached to the bedrock, producing a considerable outflow of water. This type of situation is often encountered in slopes of the Carpathian flysch area, where the base of the colluvial loam layers is frequently formed of sandstone fragments and is a few metres thick. It is supplied by the groundwater from the sandstone layers of the flysch bedrock. Thus there are considerable uplift forces present in these slopes which can be reduced by drilling drainage boreholes long enough to reach the permeable base. Experience gained indicated that it is reasonable to drive the borings into the bedrock itself, since it has often been found that the discharge increased considerably when the sandstone layer was reached. In the case of Příluky the presence of the terrace gravel also contributed to the successful drainage of the slope. The important point to note is that a large slope movement was stabilized by means of just a few drainage borings.

Conversely, 11 drainage boreholes installed at the landslide near Tatobity (Bohemia) were less successful (Fig. 8–10). A thick accumulation of debris from the Permian argillites was in a state of movement caused by water flowing out of the

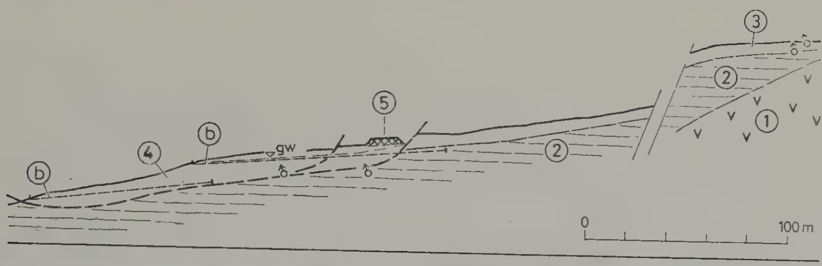


Fig. 8–10. Schematic profile of the landslide at Tatobity, Bohemia; 1 — melaphyre and porphyry (Permian), 2 — argillite (Permian), 3 — colluvial soils, 4 — slipped material, 5 — highway under construction, b — drainage borings.

bedrock. The water, however, was originating from the neighbouring intrusive body of melaphyre and porphyry, and as the latter are more permeable than the argillites, the drainage of them was desirable. But the depth of drilling required to achieve this was more than 500 m, and the drainage effect of the boreholes that were driven into the argillites was strictly local. Therefore small slides continued to occur near the stabilized areas of the slope. The low permeability of the argillites compared with that of the melaphyre was manifested by the high water level, and appearance of springs along the line of contact. The projected low road embankment was scrapped in favour of five bridge spans supported by large diameter piles.

Twenty years of experience have shown that drainage boreholes are also effective for the control of earthflows in all kinds of soils except for Neogene clay areas where it was necessary to support the toe of the slope with a supporting fill.

An example is given by the sheet slide that disturbed the slopes of Petřín hill in Prague in 1965. The movement, which destroyed the embankment of a funicular

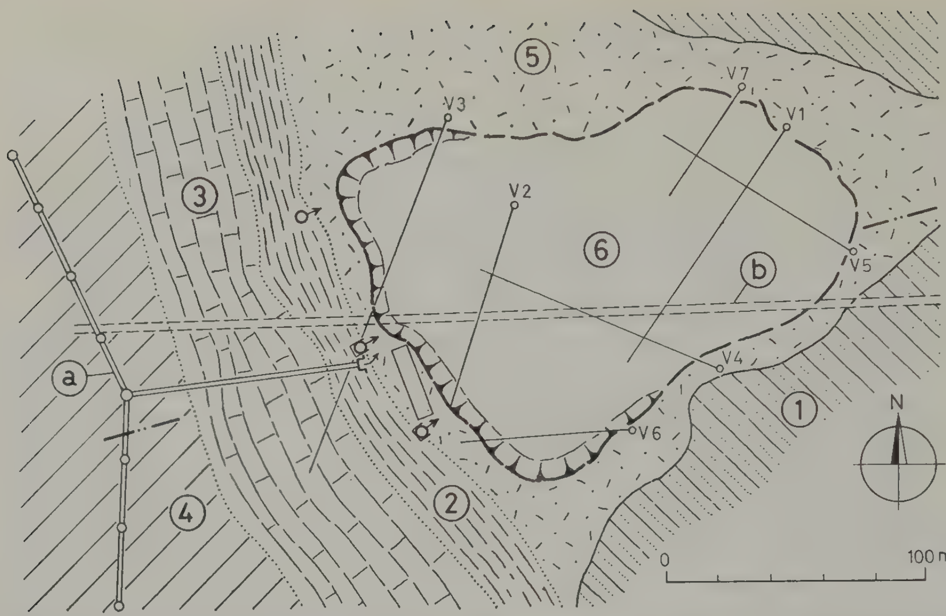


Fig. 8–11. Sheet slide of slope debris on the slopes of Petřín in Prague; 1 — Ordovician shale, 2 — Cenomanian claystones, 3 — Cenomanian sandstones, 4 — Turonian sandy marls, 5 — slope debris, 6 — slipped slope debris, a — drainage gallery with drainage boreholes directed upward, b — disturbed embankment of the funicular railway, V — drainage boreholes.

railway, involved slope debris and the weathered products of Cretaceous sandstones and claystones filling a depression in Ordovician shales, which had formed along the Prague Fault (Figs. 8–11 and 8–12). The slide surface partly followed the zone of pulled-out Cretaceous claystones and partly the surface of underlying weathered shales. The slide was about 200 m long, and in the upper region it was 130 m wide and 4–8 m deep. The head scarp formed below the level at which several springs issue forth, draining the ground-water accumulated above the Cretaceous claystones. The movement was triggered by high rainfall in 1964–1965, and by an increased output from the springs caused by leakage from sewers in the plateau above the slope.

The first task in the corrective operation was to divert the springs away from the slide area. Subsurface drainage was provided by installing horizontal borings, owing to the large surplus of free water in the sliding clayey material. The final treatment of the area was the construction of a drainage gallery to intercept an aquifer feeding the springs.

Several difficulties were experienced with Neogene rocks containing thin laminae of fine water-bearing sand or pockets of sand containing water, as are often encountered in Pliocene deposits. As already noted, sand is generally difficult material to bore through. Good results with sand were obtained in the thicker sand layers which were sufficiently permeable to yield water almost immediately after they were entered by the tip of the drill.

In several cases involving deep-seated remoulded materials of Cretaceous or Tertiary slopes the drainage scheme did not succeed because the process of consolidation was slow. The long-time effect of the drainage was evident, although not of great practical value. Also in cases involving large slopes in which ductile deformation had occurred (Figs. 4–8 and 4–11) drainage failed to stabilize the slope. Attempts to increase the resistance of clay layers in brown coal seams by drainage have also been rather disappointing (section 13.3.1).

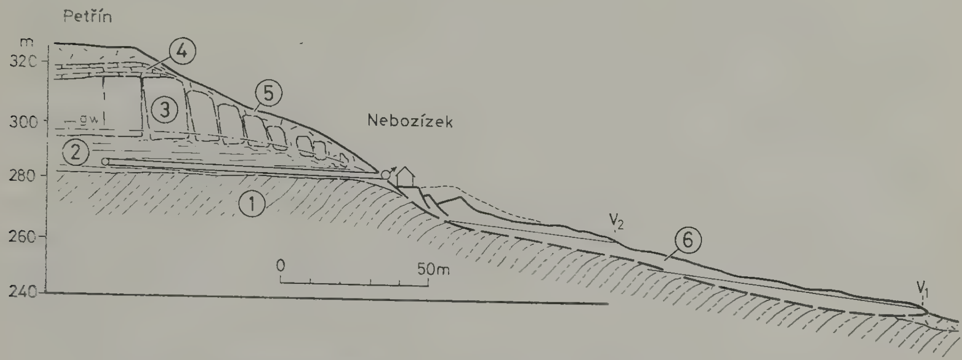


Fig. 8–12. Geological section of the Petřín landslide; symbols as in Fig. 8-11.

Where a slope is high, the effect of drainage may be reduced by the large stress dependence of the shear strength parameters; the increased effective compression may not generate the equivalent increase in shear strength, because φ' may decrease with increasing compression. This has been noted previously by Noble (1973) with respect to the residual strength. Nevertheless, Noble comes to the conclusion that "if groundwater is present, drainage should be attempted even though the prospect of removing a great deal of water may seem remote".

Short drains in low slopes can be made by driving perforated pipes into the slope. This procedure is found to be adequate where the aquifers are of comparatively thin sand or gravel beds.

Long horizontal drainage borings can be carried out by several techniques. For example, long holes are drilled with helical augers and perforated pipes are driven into the holes. Rotary drilling is done with cutter and roller bits (Jedlička and Tkaný 1965; Mencl 1965b). Perforated drill pipes also serve as a permanent casing of the bore. Through the drilling operation, the sides of the holes became packed with mud produced by the action of the drill. The bit remains in the hole. A maximum length of 300 m has been attained.

The effective life of the drain is generally affected by three factors, namely clogging, frost, and rusting. If it is evident that an otherwise active borehole has been clogged, then it can be cleared by washing. Boreholes draining downward are preferred wherever possible. In order to avoid rising water in boreholes, junction wells may be bored into the permeable layer at depth, and the wells are then drained by drilling horizontal boreholes. A row of five to seven wells is necessary to ensure that contact is made by the boring (Flimmel 1977). Alternatively, an open pit can be dug and provided with a fan of drainage boreholes.

The exits of boreholes may become blocked with ice with the result that water flowing from within the slope saturates and weakens the slope surface. The best solution is to cover the exits with permeable gravel 1.5 m thick. As far as rusting is concerned, experience shows that 30 years is a reasonable expectation for the useful life of the tubing. In order to prolongate the life, perforated synthetic piping may be inserted into the borehole after the drilling has been completed.

An integrate correction of a large landslide may be exemplified by stabilization of the Kamenose landslide in Japan (Japan Society of Landslides, 1977).

The Kamenose landslide was not the first to have occurred in this area. The first disastrous landslide recorded of 1918 buried the Yamato river valley and destroyed the railway tunnel at the foot of the valley slope. In 1967 the slope movement was renewed at a great scale and was brought under control by means of extensive surface and underground remedial measures by 1974 (Fig. 8–13). For monitoring the rate and direction of the movement, both at and below the ground surface, the air-photo survey, extensometer, tiltmeter measurements, and pipe- and strain gauging were used.

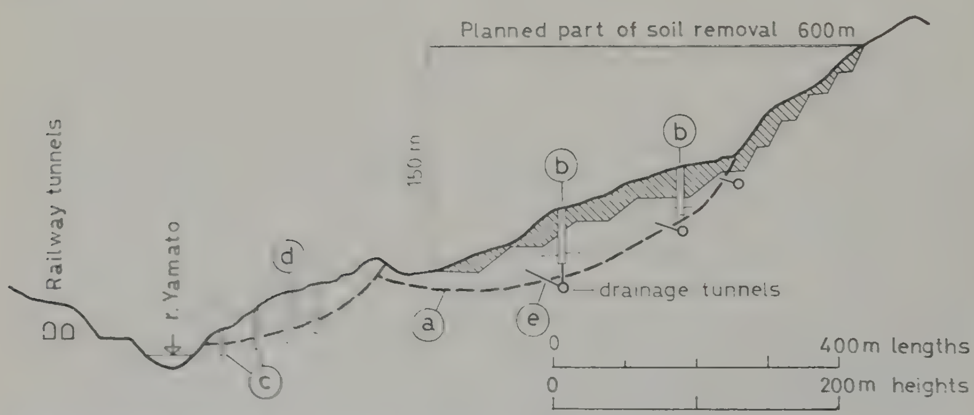


Fig. 8–13. Profile showing the removal operation and drainage tunnels at Kamenose landslide, Japan; a — slip surface, b — drainage well, c — steel pipe, d — deep foundation, e — drainage borehole (Japan Soc. of Landslides, 1977).

The area is composed predominantly of andesite lava sheets and pyroclastic rocks of Miocene age. The main slip surface extends at a depth of 40–80 m, at the level of the clayey altered zone. The slipped masses had a volume of about 22 million m^3. The landslide movement is assumed to have been brought about by the uplift of the ground-water table, which shows very great fluctuations depending on the precipitation amount.

The control works were therefore first directed to the drainage of the slide area. The measures taken involved surface drainage works, drainage tunnels and wells with drainage boreholes. Horizontal boreholes of a total length of 36 km drained the water into five drainage tunnels, which were driven below the level of the slip surface. The tunnels had a diameter of 2.10 m and a total length of 4,230 m. Sixteen drainage wells (3.5 m in diameter) and surface drainage works 3,500 m long were performed.

Additionally, steel tube piles were sunk in the lower part of the slope, and 1,000,000 m^3 of material was removed from the head area to reduce its loading. The control works have proved to be satisfactory, as the slope seems to be fairly stable.

8.4 Stabilization of landslides by planting

Slope movements generally disturb the vegetation cover, including both the tre-growth and grass cover. Reforestation of the slope is an important part of any correce tive treatment; it is carried out during the later stages of the work, invariably after at least some degree of stabilization of the landslide has been achieved. The planting of forest trees is preceded by drainage of the slide area, levelling of the surface and the filling of cracks.

Tree planting, however, is an effective method of control only in the case of shallow sheet slides. Landslides with a deep-lying slide surface cannot be stopped by development of the vegetation, although this does have the effect of reducing infiltration of slope by surface water.

It is generally accepted that forest growth helps to dry out the surface layers and that ramification of the root systems assists the consolidation process. Since trees draw water from the surface beds, the most suitable species for planting on sliding slopes are those that have the largest consumption of water and the highest transpiration rates. Thus for example, it is more advantageous to plant deciduous trees than to plant the conifers which have a comparatively low rate of transpiration.

Sýkora (1961) systematically studied the influence of vegetation on sliding movements, and suggested that in selecting suitable trees for sliding slopes, their effect on the structure of the soil should also be taken into consideration. For this reason spruces are not recommended since they have shallow root systems, and because of their relatively rapid growth they produce the most rapid increase of the loading of the slope. Sýkora recommends sowing suitable species of grass first of all, then herbs and hedges, followed finally by trees. According to the results of his investigations, the main types of indigenous forest on Czechoslovak slide areas consists of oaks with hornbeams.

Experience has shown that a mixed forest of broad-leaved trees such as oak, hornbeam, ash and alder, which may also be grown as coppice in a 30–40 years rotation, is most appropriate for the afforestation of sliding areas. Extensive clear felling should be avoided, because it disturbs the stability of the slope by bringing about changes in the surface and ground-water system; if the surface is affected by erosion, the possibility of the infiltration of surface water is greatly increased. Infiltration is also facilitated when the slide area is used as pasture; cattle tear up the grass cover reducing the degree of surface drainage, and cause damage to growing bushes and trees.

The significance of the role of plant growth in the stabilization of sheet slides is often underrated. Yet it should be borne in mind that although a grass cover prevents drying of the immediate surface and reduces the development of shrinkage cracks, it causes a net removal of water from the upper layers of the ground. Measurements of the water content of clayey soils carried out by Felt (1953) has shown that below grassed areas (Fig. 8 – 14) the water content is reduced down to a depth of about 2.5 m, and the influence of bushes reaches deeper than 3 m.

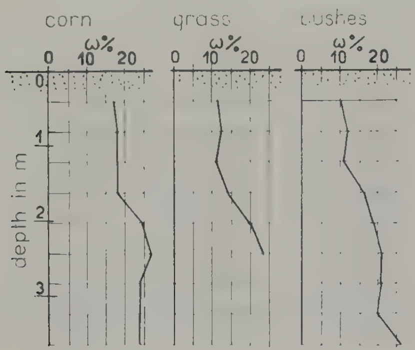


Fig. 8–14. The water content of clayey soils is diminished down to a depth of 2.5 m below a grassed surface; the influence of bushes reaches deeper than 3 m (Felt 1953).

8.5 Retaining walls and similar structures

Retaining walls are sometimes erected to bring greater stability to dangerous slopes, or to support existing landslides. They have been superseded in many cases by pile walls, which can be installed in advance of the excavation work. The construction of retaining walls requires a great deal of manual and skilled work, as well as expensive planking and the maintenance of access to the unfinished cutting.

Probably four types of walls and similar structures can be distinguished from the points of view of function:

(a) Low walls supporting slopes in clayey soils are primarily intended to prevent loosening of the toe of the excavated slope, and to protect it against frost action. As explained in section 8.2, the authors have advocated the use of sand berms. Sand berms, however, require space, and if this is limiting, as for example in the modernization of old railway cuttings, concrete walls may be used instead. Compared with pile walls, retaining walls have the disadvantage that their resistance is a function of their weight only (hence "gravity walls"). Thus it is not easy to satisfy the postulate that the resultant of forces acting on the wall should act on the middle third of the base of the wall, and therefore, walls have tended to be designed as low as possible. However, even if drainage is provided at the rear of the wall, the ground-water cannot be substantially lowered behind a low wall. The unfavourable effect of frost then causes deterioration of the slope, as shown in Fig. 8–15. The authors have several times assisted with the installation of deep drains behind low walls, but the results have not

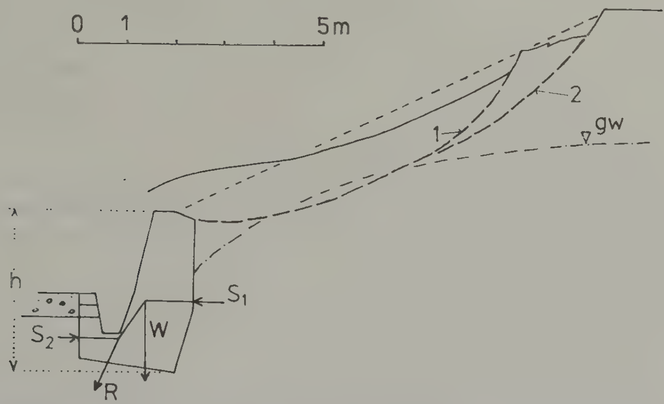


Fig. 8–15. Collapse of the slopes of a cutting near Čebín (Moravia); the slopes are of Neogene fissured clay, and the collapse was caused by frost action; 1, 2 — two successive stages of failure.

Fig. 8–16. Concrete crib walls, the Tauern Mts., Austria (photograph by Rybář).

always been satisfactory. Low walls often tend to be relatively thin, and frost can penetrate through them. This is another reason for advocating the use of sandy supporting berms, because these can be built up higher.

There is a tendency to use *precast structures* for the building of retaining walls on treacherous grounds, but because these structures are commonly thinner than the classical gravity walls, they are less resistant to frost. Crib walls, which are filled with sand, overcome this problem and have the additional advantage of being flexible (Fig. 8–16). They can also be built up on top with more sand in order to protect the higher parts of the slope.

(b) High retaining walls have to resist considerable horizontal forces with the intention either of preserving the stability of an intact slope, or of supporting a deep sliding mass. In the former case the wall has to resist the pressure at rest, which if the height of the wall is *h* metres, and the gradient of the slope is 1 in 3, reaches a value of about $S_0 = 200\,h$ (kN). This can be found from Fig. 8–17, in which the isolines of σ_x were plotted for the three cuttings of 10, 13.5, and 16 m depth, respectively (coefficient of initial lateral pressure $K_0 = 1$). As can be seen, the differences in the depths of the cuttings are only of secondary importance. Also, the ground-water table does not seem to have any great effect on the pressure. Since the above value of S_0 corresponds to a state of rest of the slope, some decrease in this value is to be expected during excavation for the wall. The decrease, however, will be smaller at higher magnitudes of K_0. With $K_0 = 1.5$ the above magnitude of S_0 is by about 50% greater.

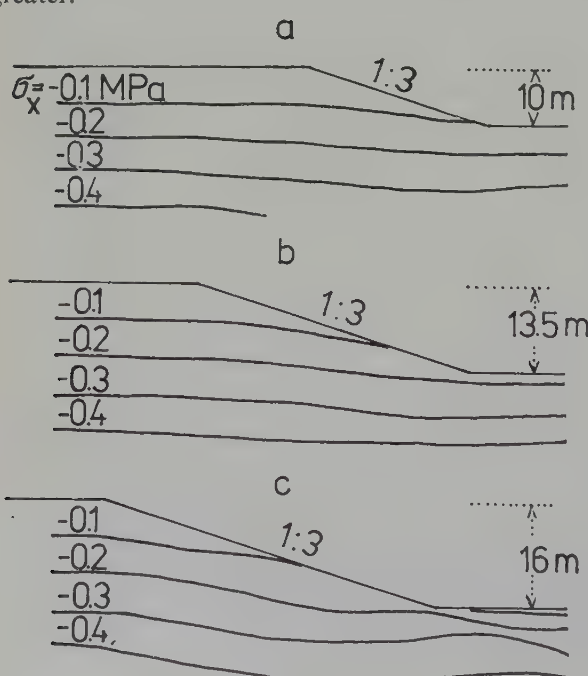


Fig. 8–17. Isobars of the horizontal compression stresses which developed in slopes after the excavation of cuttings 10 m, 13.5 m, and 16 m deep. The coefficient of the intrinsic lateral stress is $K_0 = 1$. The material is stiff clay as in Fig. 4-1 (Mejzlík and Mencl 1975).

The case of the cutting at Bánovce in Slovakia is illustrated in Fig. 4–12. A horizontal force of about 700 kN was acting on the 4.5 metres high retaining walls. The force was calculated on the basis of the stability of a 6 m deep rock wedge squeezed up at the bottom of the cutting before stabilization work began. Therefore the force cannot be considered as the effect of the pressure at rest.

Forces of such magnitude cannot be resisted by ordinary retaining walls, and therefore the walls have been integrally tied to one another to form an invert frame. The frames (Mencl, 1962) were set up in sections 7 m long and it was necessary to construct them section after section. As soon as the length of the excavation pit reached the length of three sections movement of the sliding mass was revived, in spite of the fact that the ground-water table was lowered by pumping from 16 deep wells. A survey carried out during the construction showed that the central section of the frame belt had been shifted about 30 cm to the right. Seven horizontal drainage boreholes were drilled to a length of 70 m to take over the function of deep wells, but the total discharge from the boreholes only amounted to 48 litres per minute and decreased over the next two years. It is noteworthy that the slope remained dry for several weeks following the collapse, and then surface water began to penetrate into the head scarp (Fig. 4–12).

(c) Walls may be constructed in the rock cuttings of roadways in order to protect roads from rockfalls. There is plenty of scope in these situations for designers to display their engineering skills. In the authors' practice preference was given to crib walls filled and protected with rock debris. These walls have not only function as rock traps, but also stabilize the toe of the slope which is otherwise exposed to the influences of water and frost. According to the studies by Piteau and Peckover (1978), the width of the impact area for rock faces up to 10 m high is about 3.7 m, and 6.1 m for rock walls 20 to 30 m high. A wider space is necessary if boulders are able to pick up a horizontal velocity by rolling down the sloping area above the excavated slope.

(d) Retaining walls may be employed to stop the movement of sliding masses. In these cases several alternative types of supporting structures can be adopted, from a simple buttress fill (Fig. 8–3) to rigid concrete walls or rows of pile walls. When designing the latter structures, it is necessary to bear in mind that they will be exposed to the passive earth pressure. If other conditions are the same, this pressure will be greater where stiff or plastic clayey masses are concerned compared with the pressure of mud flows (see also section 8.7).

8.6 Rock bolts and rock anchors

The technique of stabilizing rock masses with rock bolts and rock anchors which has found widespread application in mining and tunnelling, has been adopted for the prevention of movement in rocky slopes and the stabilization of rock slides. It is now also applied to the stabilization of soil slopes, in association with retaining structures.

Rock bolts are shallow-fitting elements, while anchors are fixed deep into the slope. Thus rock bolt helps to stabilize the slope face by exerting a force which compresses the joints and prevents loosening by freezing. It also makes for greater stability by functioning as a dowel, i. e. increasing the shear resistance along the joints. The bolts are mostly grouted in along their entire length with cement or other chemical agents. A prestressing treatment may or may not be given. If there is a possibility of slipping along smooth, even surfaces, pretensioning is necessary. On the other hand, "passive" bolts placed in irregularly jointed rocks begin to be tensioned automatically by dilatancy as soon as the rock is strained.

The increase in volume (dilatancy) of the rock during the shear straining is essentially the process that is impeded by bolts and anchors. Without prior dilatant straining

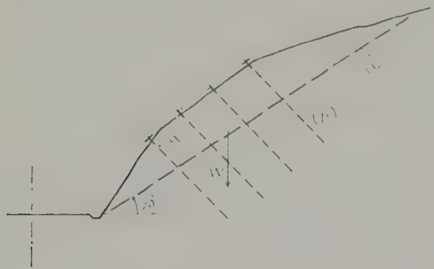


Fig. 8–18. Stabilization of a slope with prestressed anchors; a — bedding plane, b — prestressed anchor.

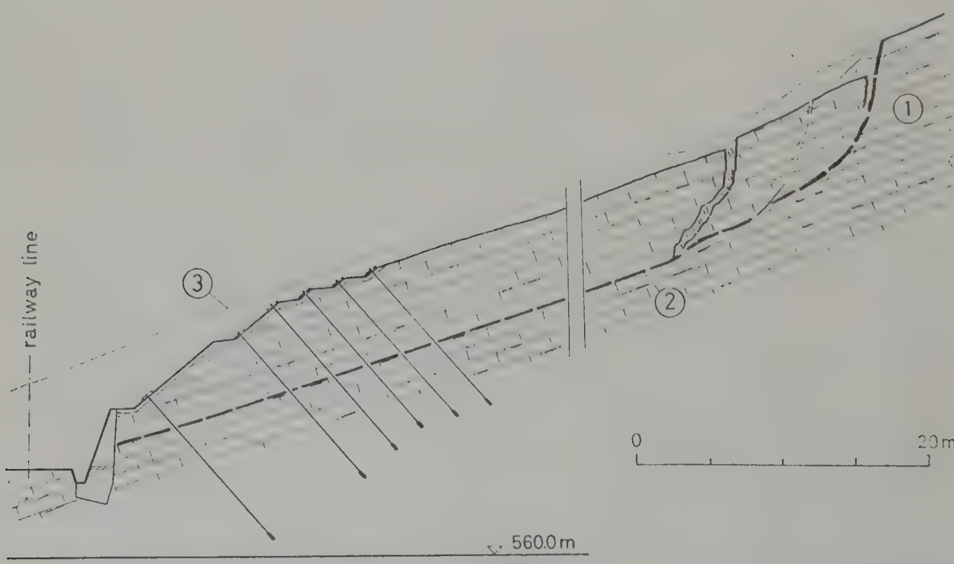


Fig. 8–19. Correction of a slide in a cutting of the Podolinec-Orlov railway (Slovakia); 1 — Palaeogene sandstones and claystones, 2 — bedding plane on which the movement took place, 3 — original ground surface.

the failure cannot develop. Rock bolts are therefore most effective in dilatant rocks, provided that the heads of the bolts are not drawn into rock face. The time of installation of rock bolts is important; they should be in placed in the rock as early as possible before the next step of excavation is carried out.

Rock anchors are longer than rock bolts. They are anchored into the rock along their root section in order to transmit a tension force to the rock face. But self-tensioned anchors, grouted with resins along the entire length, have also been applied, when dilatant behaviour of rock was considered. A detailed description of the techniques of anchoring has been given by Hobst and Zajíc (1977).

The angle of inclination of bolts and anchors should be examined carefully to ensure that if deformation of the slope occurs there will be an increase of tension in them. When the dip angle of the potential slip surface is known (Fig. 8–18), the static effect of bolts and anchors is given by

$$F = [(W \cos \beta + K \cos \alpha) \tan \varphi - K \sin \alpha]/W \sin \beta ,$$

where F is the safety factor, and K is the total force produced by rock bolts or anchors. The phenomenon of relaxation may reduce the effect of anchors. If the infilling material of the bedding planes or faults is clayey in character, tan φ can decrease to

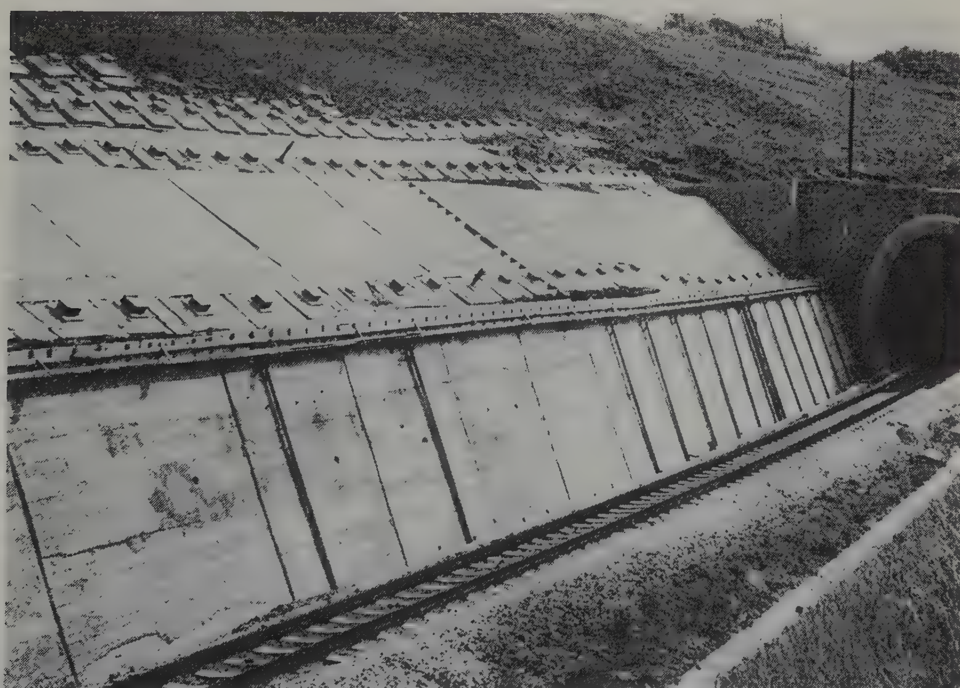


Fig. 8–20. Anchorage of the slope of a cutting of the Podolinec-Orlov railway (Slovakia; photograph by Nemčok).

about 65% of the nominal peak value, and in order to prevent yielding of the slope along such a surface, it would be necessary to prestress the anchors until a value of F greater than unity was obtained. Tests carried out with cables anchored in the Hustopeče Marl and pretensioned to 540 kN showed a relaxation of 9% over 200 days (Kosek, 1979).

A photograph of the anchoring of a rock slope in the approach cutting of a tunnel on the Podolinec — Orlov railway line is shown in Fig. 8—20. Beds of Palaeogene age inclined towards the cutting yielded after partial excavation of the cutting, and moved downwards. The angle of dip of the beds was 21°. The slippage has been prevented by means of cables 60 mm in diameter and prestressed to 1000 kN (Fig. 8—19).

Fig. 8—21 shows an example of slope stabilization in the Tertiary marl and marly limestone forming the slope above the highway near S. Remo in Italy. The sliding movement occurred along a bedding plane, dipping about 45°. The angle of shear strength measured along the planes was 21°. The thrust of the sliding mass about 6500 kN was resisted by nine concrete piers, 30 m deep and 24 m (centre to centre) apart, and by about 300 anchors 45—75 m long, each presenting an operational pull of 1200 kN.

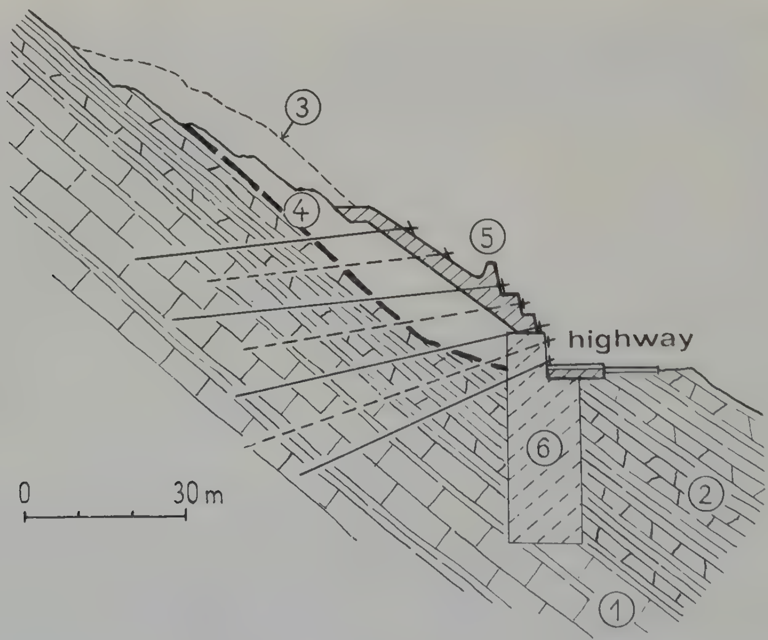


Fig. 8–21. Stabilization of a slope above a highway near S. Remo in Italy using anchors and concrete piers (Baldovin and Fattore, 1974); 1 — sandstones with marls, 2 — Tertiary marls and limestones, 3 — original slope surface, 4 — slip surface, 5 — reinforced concrete slab, 6 — concrete piers.

Walls of piles have often been used in place of retaining walls. The main advantage of piles is that they can be installed prior to excavation. Little space is needed and therefore the amount of excavation work required is reduced. Piles decrease the danger of slope movements in cuttings and provide an effective means of stabilizing existing landslides. Their construction requires less manpower, but they are relatively costly, and expensive temporary service roads are required to transport the heavy equipment.

The range of application of pile walls is limited by their bearing capacity. They are not strong enough to hold back the bottoms of deep cuttings in ground in which large horizontal stresses are present (Fig. 4–12). The horizontal force of about 7000 kN acting on the inverted frame described above could not be resisted by piles. In this type of situation, however, piles can provide short-term bracing of the excavation, which then must be strengthened as soon as possible with a thick floor slab and bracing beams.

Sheet-pile walls are rarely used, except where a temporary retaining wall is required to stop a spreading mass at the foot of a moderately sized earthflow. The greater part of the mass is retained behind the wall and that which overflows the top of the wall can be removed. The height of a sheet-pile wall is limited, 3.5 m being the greatest height known to the authors; such a wall may be considerably deformed but can still resist a passive earth pressure of about 250 kN. The other situation in which sheet-pile walls are used is in stopping shallow slab slides of slope surfaces caused by frost action.

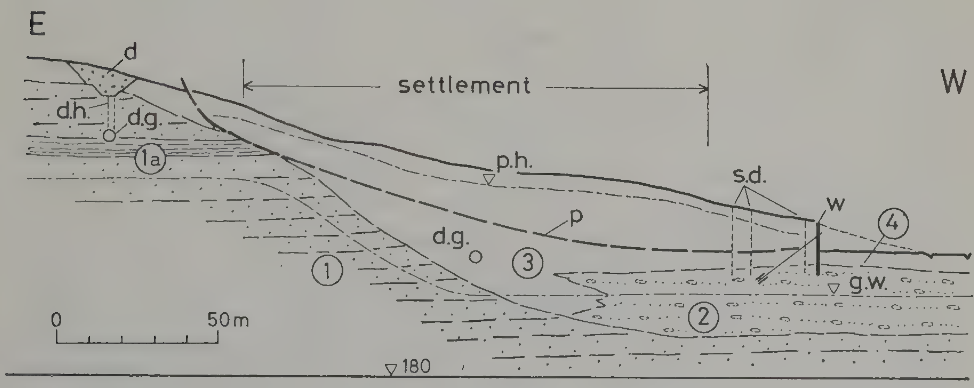


Fig. 8–22. Profile of a slope at Weirton, West Virginia; 1 — bedrock (horizontally bedded shales and sandstones, Pennsylvanian); 1a — claystone layer, 2 — sand and gravel, 3 — colluvium (the upper part having slipped down in the past), 4 — Holocene deposits, p — potential slip surface, d — drainage trench, d. g. — drainage galleries, d. h. — drainage holes, p. h. — piezometer head, s. d. — sand drains, w — anchored sheet-pile wall.

Where sheet-pile walls are used, a method has to be found for draining the ground-water which collects behind the wall.

The adoption of sheet-pile walls as permanent supports for large unstable slopes is comperatively rare. An example of this was described by D'Appolonia et al. (1967), and is shown in Fig. 8–22. An excavation was to be carried out at the toe of a slope consisting of the partly clayey and partly nonplastic colluvial soils that derive from Carboniferous bedrock. Careful investigation established the presence of high local ground-water levels with a low general ground-water table. Remedial action was taken on three fronts: draining of the slope was achieved by using two drainage galleries extending from access shafts, sand drains were laid to drain the higher pockets of water into the underlying gravel, and an anchored, retaining sheet-pile wall was constructed. When the excavation depth was no greater than 5.5 m anchorage was not necessary, and the wall did not suffer deformation. The use of ties made from steel H-beams, spaced 2.7 m between centres, made it possible to lower the ground surface by 12 m in front of the sheet-pile wall. The measured tension in the beams amounted to 675 kN, the calculated one was 945 kN. The ties were prestressed to about 490 kN.

In principle, pile walls serve either as retaining and revetment walls in cuttings to prevent sliding of the slopes and grounds behind them, or as retaining walls at the front of creeping or sliding masses. Nowadays, pile walls consist of bored piles, and the extent of their application is growing owing to progress made in boring techniques. It is now possible to install piles under adverse geological and hydrogeological conditions, in addition to fabricating piles of larger diameter. Diameters between 40 and 120 cm are most frequently used.

The designs and static calculations for pile walls do not come within the domain of the engineering-geological activity. Nevertheless, a general information on the horizontal bearing capacity of piles is necessary, because the wall should be sited in the suitable position; this means that the wall should not be overstressed, when sited towards the middle of the sliding slope. The reasonable position is near the toe, in order to take advantage of the resistance of the soil along the slip surface.

Thus, for example, piles 40 cm in diameter cannot resist a horizontal force greater than about 100 kN. The acting force can be derived by the slice-method and is the force acting at the boundary between slices coinciding with the position of the wall. Experience gained with a wall 800 m long (this wall is illustrated in Fig. 4–14/c, the piles are spaced 40 cm between centres) has confirmed the above value. With a height of 3 to 3.5 m, the wall was expected to resist a force of about 80 kN exerted by the saturated and disturbed clay sheet, the assumed angle of residual resistance being 11°. In spite of the greater force of the passive earth pressure when the soil overflowed it, the wall stood up well. The problem of the allowable horizontal force is much the same where piles of large diameter are involved. By analogy the admissible horizontal force (calculated by using the slice method), is about 200 kN for piles 80 cm in diameter, and about 400 kN for 120 cm piles.

In order to reduce the loading of the wall as much as possible, other measures need to be taken together with the erection of a pile wall. Such measures may include anchoring of the wall, drainage of the soil or some levelling of the slope.

During the construction of a highway near Hustopeče, Moravia, a cutting had to be made in the notorious Oligocene silty marls of the area. These had been cause of many problems particularly with regard to the reconstruction of nearby railway cuttings 50 years previously. Although the water content of the marls is only 15% (the consistency limits are 15% and 30%), and in spite of the normally low groundwater table (Fig. 8–23), they invariably give rise to slope movements, because surface water penetrates deep owing to the fissured structure of the marls. The shear strength parameters of saturated samples have been found to be 0.005 MPa and 21°.

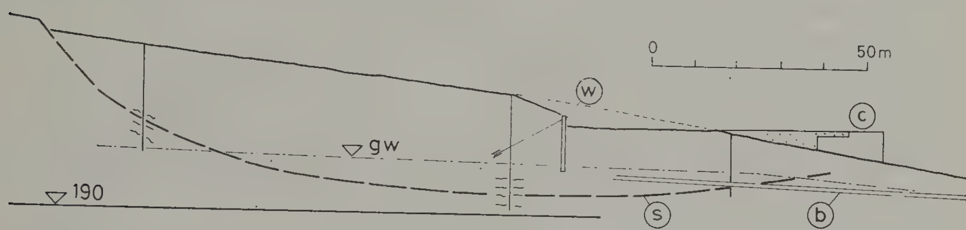


Fig. 8–23. Profile of a slope at Hustopeče in Moravia, cut in Oligocene marl for the construction of a highway; c — wine cellar, s — fossil slip surface, b — drainage borehole, w — anchored pile wall.

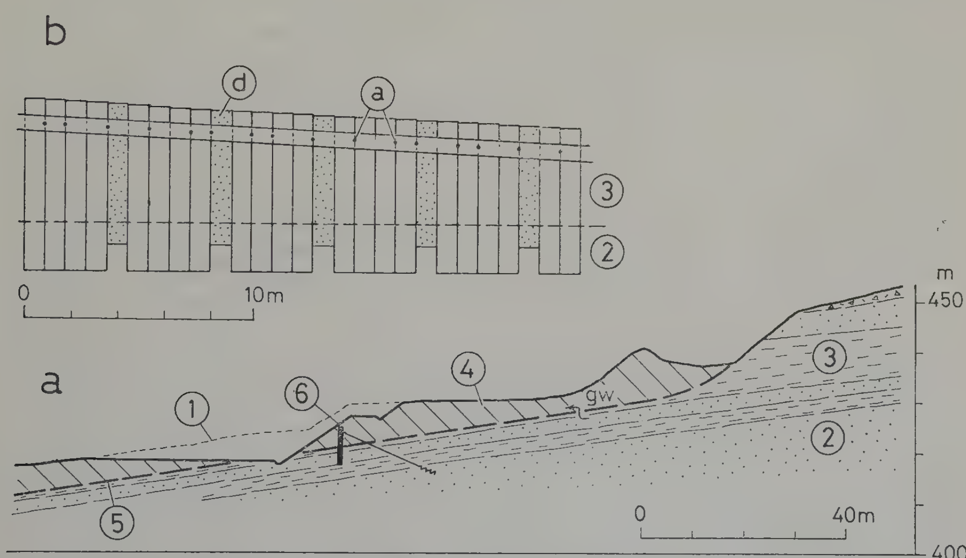


Fig. 8–24. An anchored pile wall was used to perform the additional function of a drain in a cutting for a highway near Neuenhof Baden, Switzerland; 1 — original surface, 2 — sandstone, 3 — marl (Tertiary molasse), 4 — displaced mass, 5 — slip surface, 6 — anchored wall, b — detail of pile wall, a — anchors, d — drainage piles (after Schindler and Gautschi, 1968).

The highway was situated so as to minimize the amount of excavation required. The 9 m high slopes of the cuttings were provided with walls up to 6 m in height. These consisted of piles 90 cm in diameter and 13 m long spaced 1.15 m between centres. In those sections higher than 4 m, the walls were anchored with 500 kN prestressed cables attached to alternate piles. Five deep horizontal boreholes were installed at the bottom of the slope to ensure effective drainage in wet conditions. The boreholes were placed so as to cross the south-easterly dipping beds as much as possible.

The drainage by means of horizontal boreholes of a mass of material behind a pile wall is difficult because the holes must cross the pile wall below the level of the road. Gaps are often left in the wall for this purpose. An interesting solution to the problem of the drainage was applied in one of the cuttings of a highway at Neuenhof Baden in Switzerland (Fig. 8–24), (Schindler and Gautschi, 1967). At intervals of two to five piles, an ordinary pile was replaced by a drainage pile made out of highly porous concrete.

An example of the use of piles as an immediate means of controlling a sliding slope in a railway cutting is shown in Fig. 8–25. The line runs through a cutting 8 m deep, excavated in Neogene marly clays. The slope has a gradient of 1 : 4, because

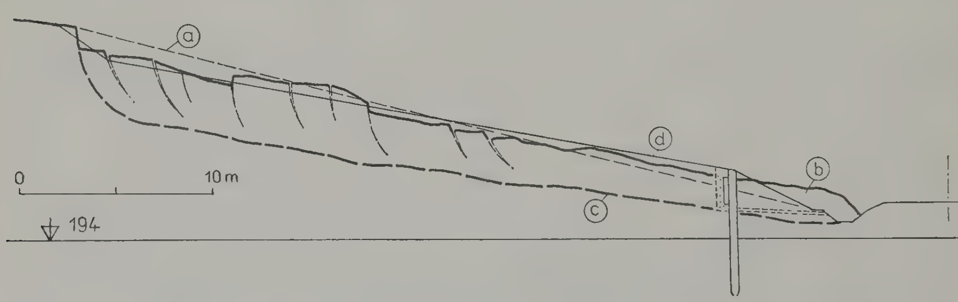


Fig. 8–25. Stabilization of a sheet slide with piles in a railway cutting near Košice (Slovakia); a — slope of the cutting before the slide, b — the slide of 1965, c — slip surface, d — regraded slope surface (after Nešvara).

the content of montmorillonite was such as to make the slope prone to volume changes. Although the ground-water table was well below the bottom of the cutting, slaking and sliding occurred, especially during the rainy spring of 1965. A sheet slide developed at the toe of the slope and extended 50 m up to the top of the slope. As the site of the slide was not readily accessible and therefore a large-scale removal of soil was not possible, piles were employed to prevent any further spreading of the landslide. Forty-two piles 6 m long were driven 4 m deep into prepared boreholes. Reinforced concrete slabs were laid against the piles (which were spaced at intervals of 1 to 1.5 m) to prevent movement of the soil between and around the piles. A sand drain was provided along the slabs. Finally the slope was regraded to make a 1 : 5

gradient. Since then the slope has displayed only shallow, slow movements which have not presented any danger either to the ditches or to the railway track.

8.8 The hardening of soils

In section 8.3, examples were cited of the stabilization of slopes by drainage. This method fails with impermeable soils, since the zone that is effectively drained is small. In these situations, methods borrowed from foundation engineering and known as hardening of soils, may be considered.

The first of these methods, drainage by *electro-osmosis*, has the same overall effect as that of subdrainage, but differs in that the water does not move towards the drainage system under the influence of gravity, being acted on instead by an imposed electric field. If two electrodes are placed in soil and a potential difference is set up between them, water is caused to migrate towards the cathode. The cathode consists of a perforated pipe; the water enters the pipe and can be removed by pumping. The method is best suitable to the drainage of silty soils with particles ranging between 0.05 and 0.005 mm. Clay particles contained in the silt harden as water is removed from the soil. The method cannot be applied to fine sand, because even if the permeability is as low as 3×10^{-4} cm sec^{-1}, gravitational movement of water overwhelms the effect of the electric current. The action of electro-osmosis may also be antagonized by the electrolytes in the ground-water which increase its conductivity but reduce the electro-osmotic flow rate of the water. At high voltages, hydrolysis of water may take place. Thus field tests are necessary to establish the parameters of the energy supply that will give the most effective drainage.

The principle of this method has been known for a long time, but the first practical application dates from World War II, when it was used to stabilize the slopes of railway cuttings (L. Casagrande, 1941).

An interesting example of the application of electro-osmosis to a stabilization problem was cited by Casagrande et al. (1961). During the driving of piles for the foundation of a bridge in Ontario, Canada, the slope of the valley collapsed. The valley occurs in glaciolacustrine deposits of silts with interbeds of sand and thin lenses of clay (Fig. 8–26). The landslide filled in an excavation for the foundation of a pier in the bottom of the valley, and destroyed the template for driving in the steel sheet piling. Laboratory and field tests indicated that a decrease in the water content brought about by electro-osmosis from an average value of 26% to about 23% would produce the increase in strength necessary to stabilize the slope. In addition, it was expected that this procedure would lower the water level in the permeable interbeds, but this effect was not considered in the static analysis. The conception of the procedure was based on the results of field tests, and consisted of treating the toe and the top sections of the slope to the full depth over the bedrock (21 m and 36 m, respectively), and the remaining section to a depth of 12 m. Four rows of anodes and cathodes were installed. The spacing between the rows of anodes to cathodes varied from 3 to 10 m and the spacing along the rows was 1.8 m for the shallow electrodes and 3 m for the deeper electrodes. The pumping pipes of the cathodes were inserted in cased holes and furnished with filters. Under the action of

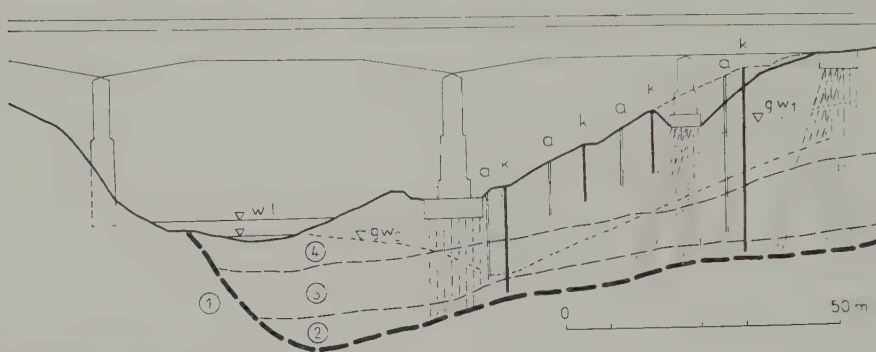


Fig. 8-26. Stabilization of a landslide by electro-osmosis at the site of construction of a highway bridge in Ontario, Canada; a — anode, k — cathode, 1 — bedrock, 2 — compact glaciofluvial sand with gravel, 3 — compact silt with sand seams, 4 — loose silt with sand seams. Intercalations of soft clay in the silts were responsible for failure of the slope; $g.w_1$ — ground-water level before stabilization, $g.w_2$ — the level after treatment by electro-osmosis (Casagrande et al. 1961).

the electric current induced at 100—150 volts, the water yield reached about 75 litres per minute. After 3 months the water content of the soil had decreased by about 4% and the ground-water level in the silty material had dropped about 10 m near the top of the slope, and about 13.5 m near the toe. The soil was stabilized to such a degree that it was possible to excavate it producing a slope gradient of 1 : 1, although the original slope had collapsed at a gradient of 1 : 2.5.

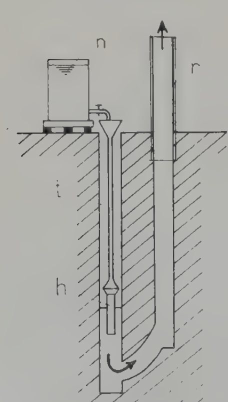


Fig. 8-27. Thermal treatment of a clayey soil (Beles 1957); n — oil tank, t — oil intake pipe, h — burner, r — chimney tube for producing air draught.

Exceptionally, the *thermal technique* is employed for the stabilization of landslides. This method was devised by Litvinov (1955) as a means of hardening loess under the foundations. The technique was used in a modified manner by Rumanian engineers for the stabilization of landslides in clay (Beles and Stanculescu, in Beles, 1957). As clay does not contain the large pores that are characteristic of loess, two holes are bored (Fig. 8 – 27) and are connected to produce a draught exhausting from a chimney tube set up at the exit. One considerable advantage of this method is that no compressor is necessary to produce an air flow. Oil is led through a pipe to a special burner installed in the borehole where it is ignited by a tissue which is laid on the upper cover of the burner and set alight before the burner is lowered into the borehole.

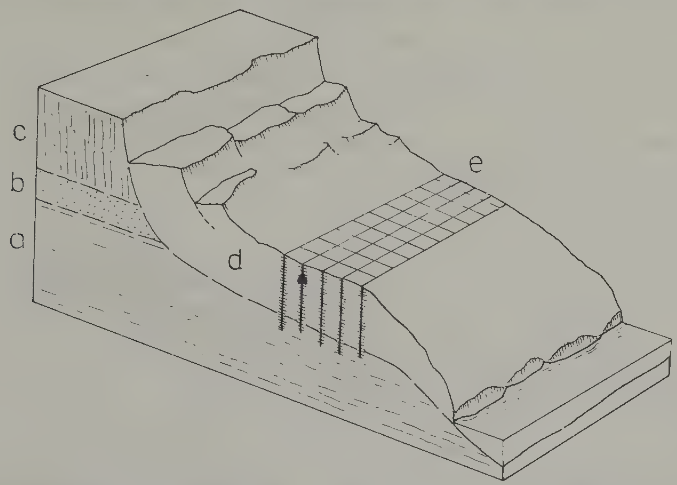


Fig. 8–28. Stabilization of a landslide near Constanza (Rumania) by thermal treatment (Beles 1957); a — plastic clay, b — sand, c — loess loam, d — displaced mass, e — thermally treated area.

Of the other methods of hardening soils employed in foundation engineering, *grouting* with portland cement is worthy of mention. This method has been used successfully for hardening the subgrade material and mud pockets underneath railway tracks. Nowadays it is also used for the stabilization of landslides along railways, as the method can be carried out without disturbing the traffic. In England, embankments and cuttings, including some in clay, have been satisfactorily treated in this way. Experience shows that the method yields good results with somewhat shallower landslides in stiff materials such as clay shales, claystones and stiff clays, which break into blocks separated by distinct fissures. Slaked material, however, cannot be grouted.

The effect of grouting is to displace water from the fissures as they are filled up with the cement mortar. The mortar hardens and creates a stable skeleton around the blocks. In principle, a mechanical stabilization of the slope is involved rather than a change in the consistency of soil mass. The grouting is begun at a pressure greater than that produced by the weight of the overlying material (i. e. 0.4 to 0.6 MPa), so that effective penetration of the suspension occurs along the fissures and along the active slip surface. At several sites, where the technique was used, a continuous layer of hardened cement mortar 6 to 12 cm thick was found to have formed along the slip surface, contributing very considerably to the stabilization of the slope (Ayres, 1961).

Before using this method a clear picture of the depth and form of the slip surface must be obtained (Chapter 6). A row of boreholes is drilled down to the slip plane and injection pipes are inserted into the boreholes. Sometimes the injection pipes can simply be rammed into the ground. According to the procedure developed in England, the grouting is carried out in successive rows running parallel with the railway track, and spaced 3 – 5 m apart. The grouting operation begins with the lowest row in order to increase the support near the toe of the endangered slope.

In Czechoslovakia the first application of this method was tried in 1961 at the railway station of Karlovy Vary (Brabec 1962; Kraus and Kubíček 1963). Tertiary tuffaceous clays overlie the Karlovy Vary granite, which has decomposed to form kaolin at the surface. The depth of the slip plane was found to be 3 to 8 m, and the stabilization process started with the drilling of nine rows of boreholes which were spaced at intervals of 5 m along the rows. The rows were 3 m apart, and were oriented perpendicularly to the railway tracks. The grouting liquid was an aerated cement suspension and mortar (Aerocem). The casing of 5 cm diameter was driven into the ground manually, and grouting pipes were inserted. The amount of cement mortar required was 2.2 m^3 per borehole, the maximum being 10.6 m^3. The stabilization of about 8000 m^3 of unstable material required 107 boreholes with a total length of 540 m, and 88 tons of cement. The grouting pressure was from 0.2 to 0.6 MPa. After grouting has been completed, the movement stopped. The lasting effect of this procedure, however, depends on whether or not ground-water collects in the grouted zone.

The mixing of lime with the soft clayey material of the slip surface is a technique that was introduced by Swedish engineers. A rotating disc auger penetrates the ground and the stabilizing agent is injected into the resulting kneaded soil column. The disadvantage of the method is that at least 60 days must elapse before the columns of stabilized soil can be subjected to loading. However good results have been obtained with slope movements of limited dimensions.

Another way of increasing the strength of clayey soils is to accelerate the process of consolidation. A loaded clay bed confined between permeable strata begins to consolidate at the boundaries of the clay. In contrast, there is little increase in shear strength near the middle of the clayey layer. When the layer is intersected by drains not only does the rate of consolidation increase, but an increase in strength also spreads more rapidly throughout the layer. Settlement of the layer occurs almost instantaneously after an increase in loading, and therefore the technique is suitable for the tretament of slopes on which it is intended to build embankments. Sand piles have been used for this purpose for many years, however, thirty years ago, Professor Kjellman developed the technique of inserting special paper filter bands into the soil.

This method has found widespread application and several modifications of it have appeared.

8.9 The treatment of slip surfaces

Taking into account the low shear resistance at the slip surfaces developing in clayey soils and weak rocks, and considering that the surrounding soil is probably capable of showing a greater degree of shear resistance, the prospect of artificially treating the slip surface arises.

The addition of lime has already been referred to in the previous section. Breaking up the slip plane by blasting it with explosives is another approach, but the use of this technique is controversial and often regarded with scepticism. No permanent effect can be achieved by treating landslides in weak plastic soils in such a manner. The method is more promising where landslides have occurred along nearly straight planes with hard rock underneath. When detonating explosives in hard rock one may expect (a) that the explosion will loosen the rock so that the ground-water uplift is reduced, and (b) that the straight slide plane will be broken up.

The prevailing opinion is that this method should not be employed for the control of deep landslides in fine-grained soils. It appears that the main disadvantage of this method is its lack of predictability.

Chapter 9

THE PREVENTION OF SLOPE FAILURES - GENERAL CONSIDERATIONS

The stabilization of slope movements is generally difficult and expensive. The indirect and extra costs are often much greater than expected; the construction programme may be interrupted with additional equipment being necessary and the entire project falling behind schedule. Moreover, landslides are always of grave concern to all involved, especially to the designers of the project. Even when no other complications develop, the moral impact is always serious, because any slope movement on a building site is a failure. Therefore the danger of a slope movement is to be anticipated; several methods appropriate for recognizing a landslide prone slope have been analysed in Chapters 6 and 7. The designer who is not capable to recognize the warnings that nature is giving should rely on the experience of the engineering geologist. Unwise is the designer who tries to do away with the cautions put forward by the engineering geologist by pointing out some findings of devious importance, for example, the presence of a clay of stiff consistency in the borehole.

A major concern of the designer should be to prevent any devastation of the natural environment. He should realize what would be the consequences of a slope failure: broken ground surface with sumps, bare hummocks, uprooted trees and disturbed vegetation – a ground that has to remain a fallow for a long time.

When the danger of a slope movement has been recognized during the projecting stage or even in the initial stage of construction, it is advisable to improve, or, if necessary, to change altogether the design. The client needs understand that the increased cost of the design will be recovered from the reduced cost of the construction work. Since the engineering-geological requirements change not only with geological conditions but also with the types of engineering work, Chapters 10 to 13 have been compiled to specify the necessary adaptations of the design.

In addition to adapting the design to the site conditions several measures are necessary to minimize the risk of slope failure. These have been discussed individually in Chapter 8, but their application poses several more problems in practice:

(a) The effect of the particular preventive measures changes with the engineering geological conditions of the site. For example, the reduction in uplift may result in a considerable increase in the stability only where the shear strength angle of the material is large enough to increase the shear resistance. Therefore, there exists a good possibility that the deep drainage will result in the stabilization of a slope composed of debris containing sandstone fragments, sand grains and a small portion of clay, as is the case, for example, in many Palaeogene rock complexes. On the other hand, drainage will not be very effective in a Neogene clay mass containing slip planes of ancient slope movements. Very often a combination of several measures is necessary. A tentative statical analysis, based on a working hypothesis and on local experience, should be made in order to decide which possible techniques are to be used and which static data are to be investigated for the purposes of the final design.

(b) The choice of the technique to be applied to the prevention of the slope movement is also influenced by several aspects of economic character. Until about sixty years ago, surplus in manpower and the relatively high cost of large equipment led to the preference of the hand-made measures. Drainage trenches deeper than 10 m were no exception. Retaining walls, built by sections, in braced excavations, remain as monuments of the past. Then came the era of heavy construction plant. Extensive earth moving operations were being carried out. It was easier to excavate a considerable part of the earth mass or to fill large stabilizing berms than to carry out the more sophisticated preventive measures. Drainage was executed by means of pumping boreholes and was mostly of temporary character. If necessary, drainage galleries were driven into the slope. This technology became uneconomical due to the urgency of reducing fuel consumption. The tendency towards subtle structures not requiring large transports starts prevailing. Massive earth berms are being replaced gradually by pile walls and when protection fills (Fig. 8 – 5) are preferred on account of their great stabilizing effect, they are filled with local earth material. Their internal drainage is provided by horizontal drains, pipes, permeable nets or other artificial units located at the contact of the fill with the underlying clay. This tendency to subtle structures will probably become more pronounced together with the need for a more refined engineering-geological investigation and a still more economical approach of the designer to the given project.

(c) The local conditions of the site play a dominating part in assuring the stability of potential slide area. For example, it is not easy to provide a permeable rock for the construction of drainage and consolidation sheets in many areas of Cretaceous or late Tertiary sedimentary rocks. An experienced engineering-geologist will look for this material in the rather coarse basal layers of the formation, or in the scanty river terraces.

(d) The importance of the project also influences the measures to be taken. A slope movement of the embankment of an important railway under reconstruction may result in higher economic losses than the failure of a slope of a secondary road. The railway designer will not hesitate to apply several measures simultaneously in order to obtain a high factor of safety, while a far more economic approach can be expected of the designer of the road. However, there is always a danger that both the designers will be censured afterwards, a risk always to be taken by those who have to consider problems in advance and not subsequently.

(e) It is also the task of the designer to draw up a time schedule for implementing the various measures to be taken; in doing this several aspects may escape the attention of the designer. For example, deep drainage made prior to excavation not only increases the stability of the slope, but also diminishes its deformation. An analysis of this problem is shown in Fig. 9–1 which considers point A of a slope 16 m high (gradient 1 : 3) in clay material, with a coefficient of lateral stress equal to unity; the original water table is situated at about mid-height of the slope.

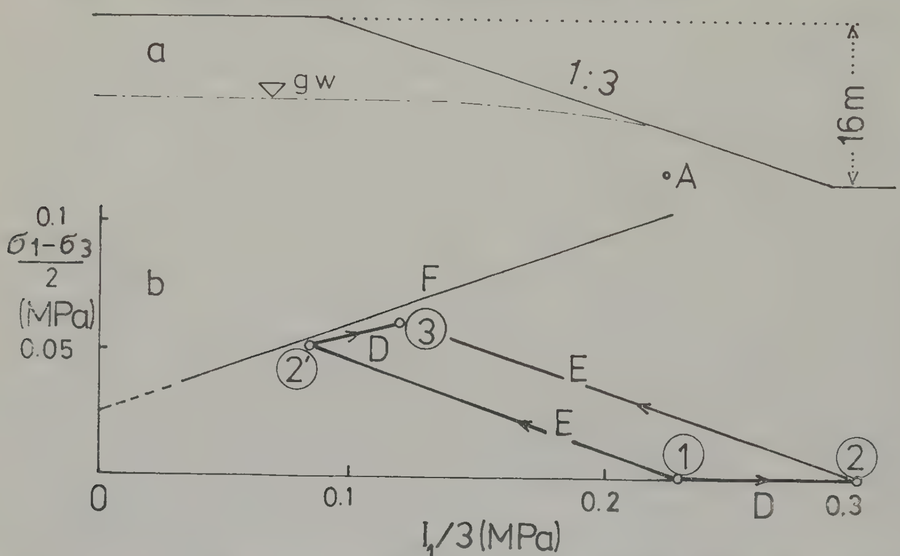


Fig. 9–1. Stress-path diagram showing changes in the stage of stress at point A (fig. a) of a slope in clay, with $K_0 = 1$. The stress conditions can develop either via the states 1-2'-3, as a result of drainage (D) following excavation (E), or via the less dangerous stress path 1-2-3, when drainage is carried out prior to excavation.

The diagram is plotted with coordinates $I_1/3 = (\sigma_1' + \sigma_2' + \sigma_3')/3$, and max $\tau = (\sigma_1 - \sigma_3)/2$ (as in Fig. 6–27). The states of stress for point A were calculated by the finite element method. The path 1–2–3 indicates the changes in the stress state if drainage (D) precedes excavation (E). If carried out a considerable time ahead of excavation the drainage may produce a consolidation process and the deformations of the slope may be reduced during successive excavation. In contrast, the path 1–2'–3 (drainage following the excavation) not only approaches the limit line F at point 2', but is also accompanied by larger deformations of the slope in the course of excavation.

Chapter 10

MASS MOVEMENTS AND DAM CONSTRUCTION

A suitable dam site should comply with two conditions: (1) a narrow cross-section of the river valley, and (2) a widening of the valley upstream so as to provide for a reservoir of large storage capacity. A constricted sector of a valley usually indicates that the slopes are built up of solid rocks that have been more resistant to erosion than the rocks in the wider parts of the valley. However, such a topographical feature is not necessarily the result of the strength of the rocks but may be due instead to the occurrence of a landslide. Even firm solid rocks are not always to be found in their original position and the narrowing of a potential dam site by a rockslide may not be recognized until geological investigations are conducted. The intended site for the Orlík Dam on the river Vltava in Bohemia, for example, turned out to be the site of a large rockslide which had occurred on the left flank of the valley (Fig. 10–1) and which was identified by Zoubek (1953). The porphyroids in the exploratory drifts showed incongruent deposition and strong loosening. The existence of an old slope failure was confirmed when test pits were sunk from the drift at the foot of the slope; gravel of the Pleistocene river terrace was found below the slipped porphyroids.

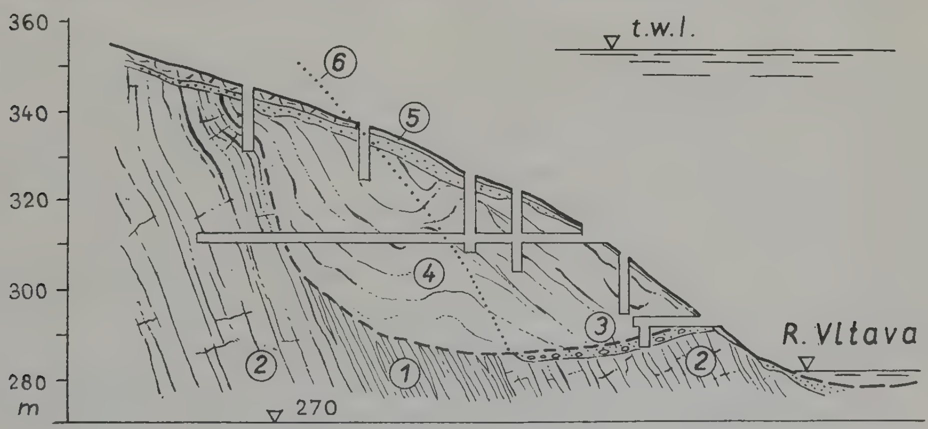


Fig. 10–1. Rockslide at the abandoned dam site on the Vltava near Zlákovice in Bohemia; 1 — porphyroids of the Jílové zone, 2 — epidiorites, 3 — Pleistocene terrace gravels, 4 — slipped porphyroid block, 5 — stony slope debris, 6 — inferred slope surface before the rockfall (from Zoubek, 1953).

Several other dam sites on the river Vltava had to be abandoned for the same reason. Frequent rockslides in the deep river valleys of Central Europe can be accounted for by the climatic conditions prevailing during the Pleistocene. Because of freeze-thaw action the joints were gradually widened and the rocks of the valley slopes were loosened. Where the loosened rocks did not fall down, they could easily be caused to do so by the excavation necessary for keying the dam.

A similar case from the French Alps is shown in Fig. 10–2 (Gignoux and Barbier 1955). Here, an apparently morphologically suitable site was investigated in the upper course of the river Romanche, where the left flank of the valley is made up of granite

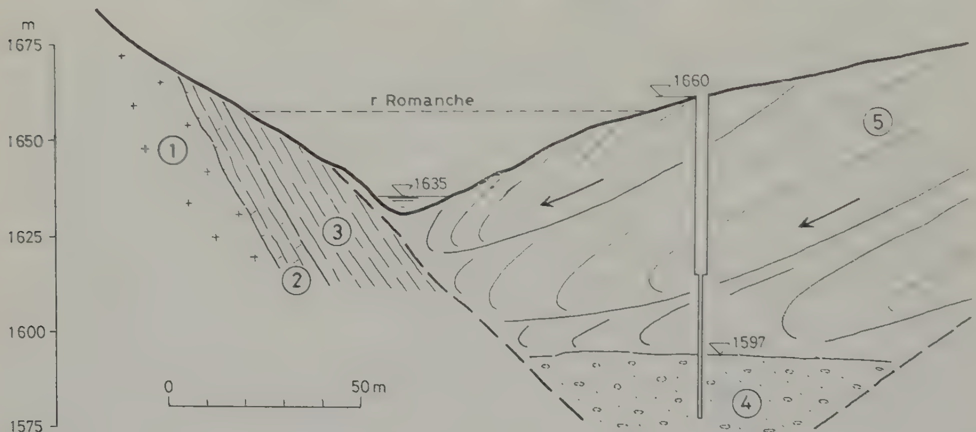


Fig. 10–2. A narrowed section of the Romanche river valley, once considered as a potential dam site, has formed as a result of the accumulation of slipped Lias shales (5); 1 — granite, 2 — sandstone, 3 — Triassic limestones, 4 — sandy gravels of the ancient valley (from Gignoux and Barbier, 1955).

and Triassic sandstones and limestones. The more moderately sloping right flank also appeared to be stable; the Liassic shales encountered in boreholes were regularly bedded and impermeable. Since a geological study of the area indicated that the valley had been constricted by an old landslide, a deep borehole was sunk in the right-hand side of the valley and revealed water-bearing gravel at a depth of 63 m. The ancient river valley had been dammed by a landslide to a height of 40 m, and the river channel had been diverted 70 m towards the left bank. The thick sequence of Liassic shales was thus not in its original place, but had slid onto the permeable alluvium of the old valley. This site had therefore to be abandoned.

The presence of displaced masses are not always an obstacle to dam construction, they may even be used after thorough examination to form part of the dam body. The situation is more serious if the site is threatened by a landslide which the geological investigation has underestimated in terms of potential danger, or not revealed at all. In such cases expensive safety measures are necessary, the construction usually being greatly delayed and the cost estimates exceeded. A carelessly planned operation has often resulted in abandonment of the site, sometimes when the work was already in an advanced stage.

The problem of slope stability has to be given due consideration particularly during the reconnaissance studies and also during the construction work when the stability may be disturbed by excavations. Stability may continue to be a problem after construction is completed, since slope movements may be reactivated by the fluctuation of water level in the reservoir; the landslides are particularly dangerous near the upstream side of the dam, where they may threaten the function of intakes.

10.1 Dam sites in valleys partly blocked by slipped masses

Large landslides and rockfalls may obstruct mountain valleys and give rise to temporary lakes, which usually vanish once the water has started to flow over the top of the barrier. A valley constricted by landslide debris can occasionally be a suitable site for dam construction and the slipped material on the valley floor may be incorporated as part of the dam body. In such cases, however, the sealing of the dam foundations and more importantly the properties of the landslide material will demand increased attention. To establish what safety measures will be required it is necessary to carry out a detailed geological investigation of a major sector of the valley and to examine the type of the mass movement and the properties (mainly the permeability) of the displaced rock material.

An experienced geologist is able to recognize an ancient landslide blocking part of a valley simply from the morphological forms of the terrain, or even from a detailed topographical map or aerial photograph. Fossil slides buried by younger slope de-

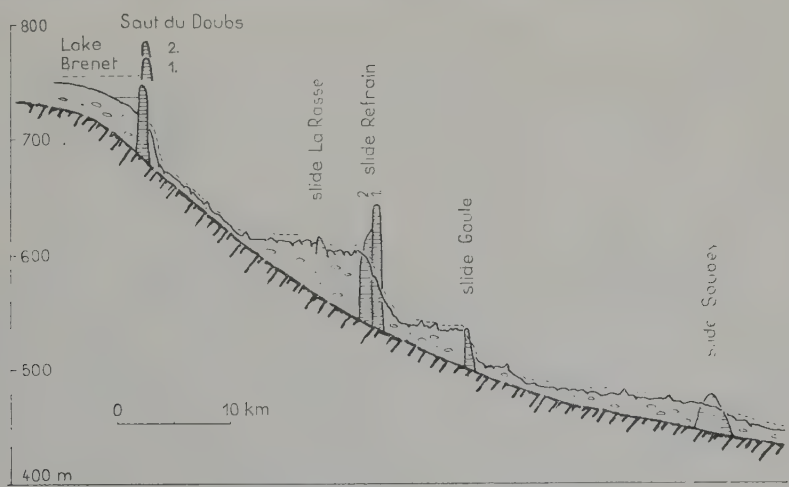


Fig. 10-3. Conspicuous steps in the profile of the Doubs river in the French Alps were caused by the damming of the valley by landslides (after Buxtorf, 1922).

posits are usually not discernible morphologically and can be disclosed only by boring, as part of the detailed investigations.

It is also helpful to inspect the longitudinal profile of the river, since sites of ancient landslides are generally manifested by striking breaks in the river gradient. In the longitudinal profile of the river Doubs in the French Alps, for example, several steps are observable which were produced when the valley was dammed by old landslides (Fig. 10–3, Buxtorf 1922).

An instructive example of how a displaced mass was used as part of a dam is given by the 20 m high Bonneville Dam on the river Columbia in North America (Fig. 10–4). The chosen dam site was some 60 km above the town of Portland,

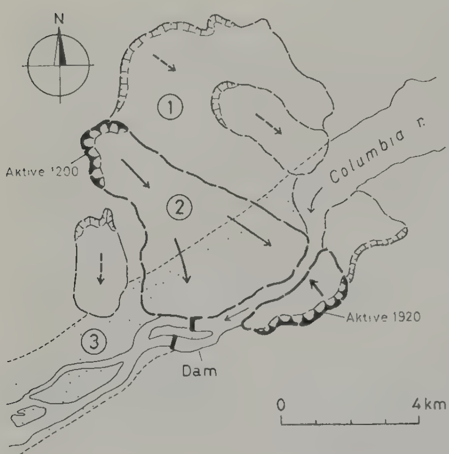


Fig. 10–4. Sketch showing the Bonneville landslide which partly blocks the valley of the Columbia River (after Palmer, 1977); 1 — Pleistocene landslide, 2 — landslide dating from the year 1200, 3 — flood plain of the river Columbia.

to avoid the effects of high tides. There the deep canyon of the river Columbia cuts through the Cascade Range for about 120 km. The resistant basalt and other volcanics rise more than 1200 m above the river and are underlain by older incompetent volcanoclastic rocks. The undercutting of the younger more resistant sequences by river and flood erosion has created the conditions for numerous slope movements.

One of these slope failures occurred at the site of the Bonneville Dam and its existence was known before the investigations began. The "Cascade Slide" caused a diversion of the river channel in this sector (Fig. 10–5), and the surface form of the slide suggests that it is the result of several events which probably began in Pleistocene times. At that time the river flowed in a deep gorge near the right bank, some 70 m below the present-day level. The total landslide area covers about 30 km^2 and the total volume is nearly 4,000 million m^3. The slid mass includes basalt blocks and agglomerates, which moved down the Tertiary "Eagle Creek Formation" composed predominantly of tuffs and silts. At present, the landslide is at rest, but basalt boulders have been observed to fall in the upper part of the slide area (Palmer 1977).

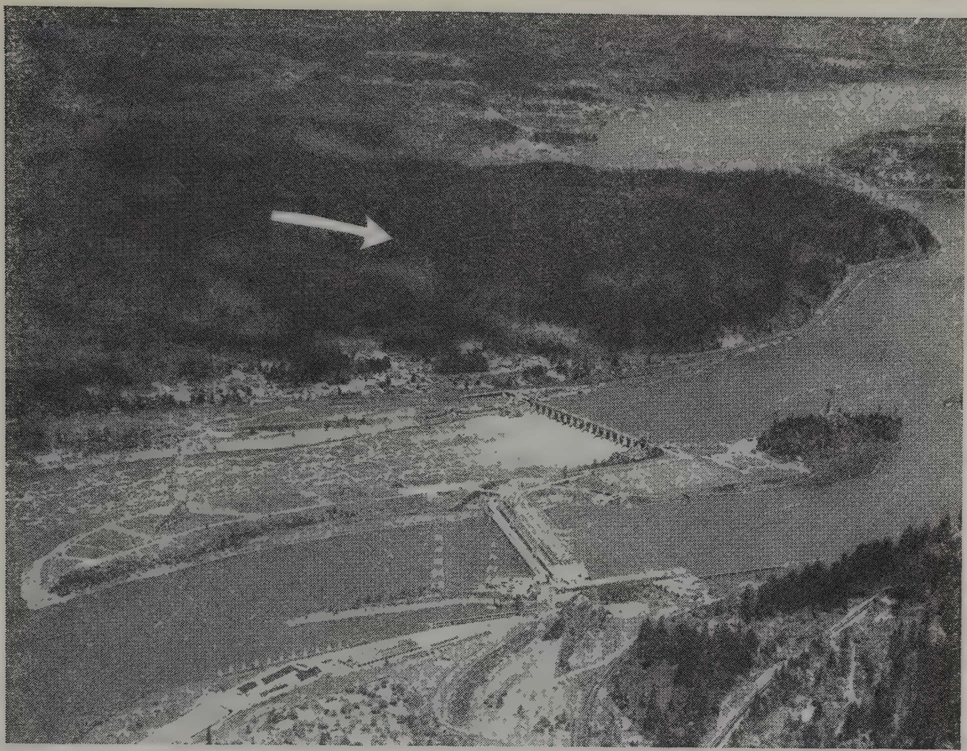


Fig. 10–5. Bonneville Dam and power plant on the Columbia River in Oregon, U.S.A. The large area on the right-hand bank of the river is the ancient Cascade slide (courtesy of W. H. Stuart).

The original design of the dam had to be modified to comply with the geological conditions of the site. The dam site was located about 100 m farther downstream so that the power plant and lock could be founded on a basalt sill, and the dam body with its spillways built upon volcanic agglomerates and sandy tuffs.

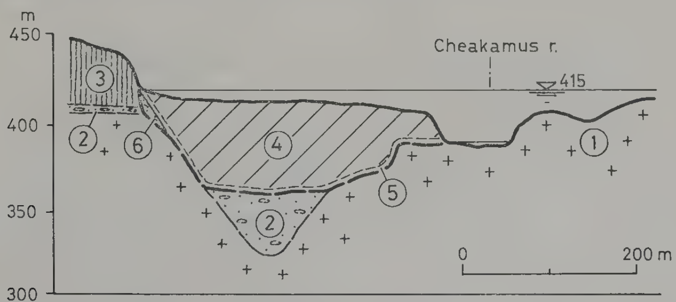


Fig. 10–6. Section through the buried valley of the Cheakamus River at the dam site (after Terzaghi, 1960); 1 — granite, 2 — fluvial sediments, 3 — basalt, 4 — landslide material, 5 — buried forest, 6 — basalt talus.

Another example is the 30 m high earth dam across the Cheakamus River in Canada, which was built partly on granite and partly on landslide material. The slope failure occurred about 100 years ago, when andesite debris blocked the Cheakamus River valley to a height of 50 m (Fig. 10 – 6). The most permeable material at the site proved to be the original ground surface under the slipped masses, where the tree trunks of a buried forest had accumulated. The gravity blocks with the spillways were erected on the granite and the earth dam was built on the landslide tongue using landslide material. The dam body and the drainage system were designed by Terzaghi (1960). The total loss of water by leakage at full reservoir capacity is about 0.019 $m^3 s^{-1}$ which is negligible under the existing geological conditions.

In contrast, large amounts of water were lost by leakage from the reservoir of the Nakhla Dam in Morocco. This is a 40 m high rockfill dam with a concrete facing on the upstream side, which was built to provide a water supply for the town of Tetouan. According to Barbier (1974), inadequate geological investigation led to the siting of the left abutment of the dam on a large landslide covering permeable river deposits. The valley is cut in a flysch sequence and the alternation of friable sandstones and clayey shales causes a susceptibility to sliding. After the construction work had been completed, heavy losses of water occurred beneath the dam, and slope movements were reactivated both upstream from the dam, owing to the fluctuation of the water level in the reservoir, and downstream from the dam as a result of water percolating from the reservoir through the permeable rocks of the left slope. The dam was in serious danger, and it was necessary to reduce the water level only to the mid-height of the dam and carry out costly remedial operation. First of all, leakage under the dam was stopped by a grout curtain under the concrete facing. At both the upstream and downstream faces of the dam, rockfill buttresses were set up in the sliding zone. Across the slide an impermeable diaphragm of cast-in-place piles was constructed parallel to the dam. At the downstream face of the dam numerous drainage borings were drilled from a drainage gallery. Stabilizing fills were also added to the right abutment, where the glory-hole spillways were threatened by shallow landslides. These measures were successful but because the project went ahead without sufficient geological investigation, the cost of the dam was enormously increased.

Geological investigations carried out for dam projects in the deep valleys of the Alps have revealed gravitational movements of mountain slopes extending to considerable depths, this phenomenon having been termed "*Sackung*" (Clar and Zischinski 1968). The movements are mostly associated with slopes formed of the metamorphic rocks (e. g. phyllites, mica-schists, graphitic slates) which show slow creeping along the planes of separation, without the formation of a continuous rupture plane in the initial stage.

Several dams have been built on these displaced blocks quite successfully, one example being the Durlassboden Dam in the Gerlos valley in the Austrian Alps. A detailed study of this dam site has shown that firm chloritic schists are exposed on the left side of the valley, whereas the right slope consists of graphitic schists and quartzites that were displaced by a deep-seated slope movement towards the valley floor.

During the Pleistocene the valley was overdeepened by a glacier and thus the bedrock was not located in test borings until the drilling reached 130 m beneath the valley floor. The valley is filled with glacial morainic material overlain by lacustrine silty deposits, which in turn are covered with glaciofluvial sand and gravel. The boreholes revealed that the downslipped block on the right-hand slope had sunk into lacustrine sediments (Fig. 10–7), which indicates that the slope failure occurred after the deposition of the lacustrine silts (Mignon 1968, Záruba 1979).

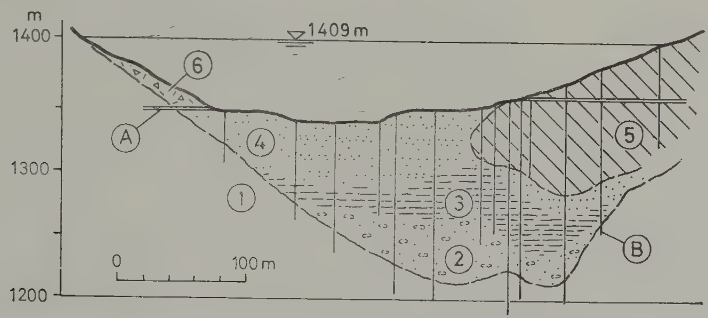


Fig. 10–7. Section through the Durlassboden dam site (after Horninger, 1958); 1 — chloritic schists, 2 — glacial deposits, 3 — lacustrine silts, 4 — sand and gravel, 5 — downward displaced block of graphitic schists with quartzite layers, A — exploration gallery, B — borings.

With regard to the intricate geological structure of the site, a rockfill dam 70 m high with a vertical impervious core was constructed there. A grouting gallery was driven in the foundation of the impervious core, and from it a grout curtain was made so as to extend into the lacustrine silts about 50 m below the valley floor.

A similar problem had to be solved by the designers of the Beauregard Dam in the Italian Alps (Fig. 10–8). Geological investigations showed that at the chosen site

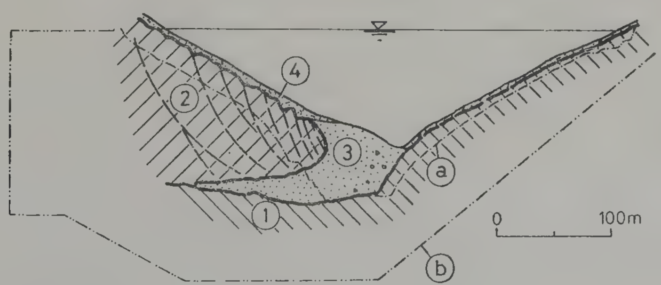


Fig. 10–8. Section through the Beauregard dam site in the Italian Alps; 1 — firm mica-schists, 2 — disturbed mica-schists, 3 — glaciofluvial sand and gravel, 4 — slope debris, a — depth of foundation, b — grout curtain (after Desio, 1974).

the valley was narrowed by a Pleistocene rockslide and that the left slope was formed of strongly disturbed mica-schists, which had slipped onto the alluvium (Desio 1973). The construction of an arch dam 132 m high demanded careful adaptation of the design to the geological structure of the site (Záruba 1974).

Gravitational creep may also cause squeezing-out of incompetent rocks in the valley floor from underneath more rigid rocks (so-called *bulging*, Hollingworth et al. 1944); as a result, block movements occur on the adjacent slopes. These phenomena have been described at several dam sites in England (Terzaghi 1950, Richey 1964, Kellaway 1972, Horswill and Horton 1976); in Czechoslovakia bulging was observed during the construction of the Žermanice Dam near Ostrava (Záruba 1956, 1958a). In the latter case the beds of Cretaceous marly shales in the valley floor were squeezed out by the weight of a layer of volcanic rock (teschenite) up to 25 m thick (Fig. 5 – 50). The teschenite blocks sunk into the shales as they moved down the slope. The steps formed by the blocks were levelled with slope debris and loess loam indicating that the main deformation dated from Pleistocene times. The rocks disturbed by the slope movements had to be sealed with a concrete wall and grouting.

Similar examples may be cited from Romania, e. g. in the valleys of the rivers Oltul and Tarnava Mare which are cut in Tertiary sediments. The bulges there are termed the "*valley anticlines*". Bulging caused by the squeezing out of underlying soft rocks and deformations of the adjacent slopes were discovered during the geological investigations for a dam on the river Arges (Záruba 1958b).

In areas of considerable horizontal stress in the surface beds, the rock in the valley floor may undergo similar deformation after the stress has been released following rapid erosion of the valley. Compression and bulging of the rocks in the valley bottom are caused by the movement of the rock mass out from the slopes and towards the valley.

Conditions such as the above were discovered by foundation explorations at the site of the Laurel Dam in Kentucky. The strongly compressed and sheared sandstones and shales in the valley bottom had to be sealed with grouting to form part of a grout curtain beneath the core of the rockfill dam (Radbruch-Hall and Varnes 1976). A similar case was observed during the construction of a dam in the Allegheny Plateau in the eastern United States (Radbruch-Hall 1978).

The main task of geotechnical investigation under these circumstances is to find out whether the rock deformation occurred in the past or whether the movement is still active.

10.2 Landslides caused by construction work at dam sites

The foundation of a dam usually requires deep excavations in both the valley floor and the valley sides, and this may disturb the stability of slopes even those of solid rock, if they are intersected by unfavourably inclined planes of discontinuity.

Thus, for example, the right-hand side of the valley above the dam near Dobšiná in Slovakia was disturbed by the excavations for the lateral blocks of the gravity dam (Fig. 10–9). A tectonically disrupted gabbrodiorite body slid down on the Carboniferous graphitic shales dipping toward the valley. The excavations for the side blocks

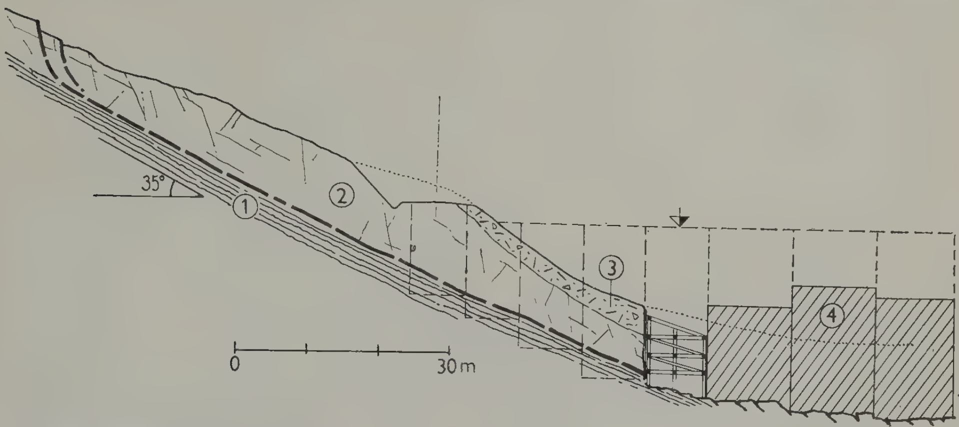


Fig. 10–9. Stability of slope disturbed by abutment excavation; 1 — Carboniferous graphitic shales, 2 — disrupted gabbrodiorite, 3 — slope debris, 4 — blocks of dam in the course of construction.

had to be carried out in successive portions in narrow shafts braced by heavy sheeting. The landslide came to a standstill after the dam had been completed and a stabilizing fill constructed at the foot of the slope (Záruba and Mencl 1976).

The building site of the Grand Coulee Dam on the Columbia River in the U.S.A. was threatened by a movement of young sediments, which was eventually stopped by freezing the sliding masses. The excavations for the foundation of the Fork Site Dam on the river San Gabriel in California caused such a large rockslide along a fault zone in granite that the building site had to be abandoned (Fig. 1–14).

The danger is particularly great in deep mountain valleys, where the slopes rise high above the crest of the dam. In these situations the slope stability may be disturbed when excavations are carried out for a cable-way crane above the top of the dam. At the Bicaz dam in the flyschoid rocks of the Carpathians in Romania, for example, the cable-way crane foundation had to be stabilized by anchoring and grouting.

An example of the effect of excavation on the disturbance of slope stability in solid rocks is given by the rockslide that was provoked by excavations for intakes at the toe of the Dalešice Dam in Moravia. The slope is composed of amphibolite and granulite which are of high strength but which are also disrupted by several fault systems. At the amphibolite/granulite contact there is a major fault zone along which the rocks have been crushed and altered into schistose mylonite. When the excavations had reached a depth of 20 m below the valley floor so that a steep wall more than 30 m high had been formed at the foot of the slope, a block of amphibolite, about

150,000 m^3 in size, suddenly moved down along the mylonite zone. The surface of separation occurred in the mylonite zone and was predetermined higher up by a course of fractures (Fig. 10–10). In the first stage of the corrective measures a temporary stabilizing berm was built at the foot of the slope and excavations for a spillway in the upper part of the slope were begun. Subsequently, the slope was further stabilized by making horizontal benches, from which anchors extending into the firm granulite were installed. At the bottom of the excavation a levelling layer of concrete was anchored to the bedrock and the stabilizing berm was gradually removed.

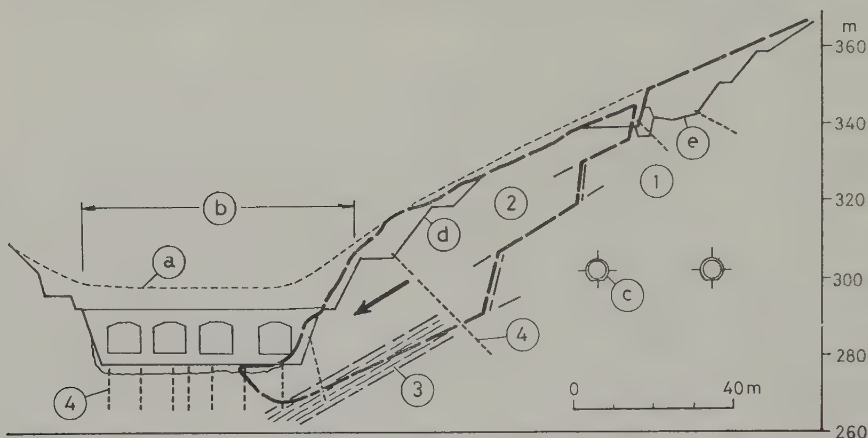


Fig. 10–10. Section through the rockslide on the downstream side of the Dalešice Dam (Mencl 1977); 1 — granulite, 2 — slipped amphibolite block, 3 — mylonite zone, 4 — anchors, a — original ground surface, b — excavation for intake tunnels, c — diversion gallery, d — slope surface after correction, e — excavation for spillway chute.

During the construction of the Tresno Dam on the river Sole near Cracow, the left slope of the valley consisting of a flysch sequence with alternating sandstones and shales was disturbed by excavations for the intake tunnel. The slipping sandstone beds were stabilized by a system of anchors and careful driving of the tunnel made it possible to construct the intake according to the design plans.

10.3 Slope movements occurring after filling of the reservoir and during operation

The filling of the reservoir and subsequent fluctuations of the water level may provoke new slope movements or renew old movements which may even occur high above the reservoir water level. Large landslides are particularly likely to occur in poorly consolidated sediments that make up rapidly receding slopes. Landslide material reduces the capacity of the reservoir, and slope failures, expecially if they occur near the dam, endanger the operation of the appurtenant structures (Fig. 10–11).

In order to assess the safety of reservoir banks, engineering geologists have to study the rocks which make up the valley sides, particularly within the zone of water fluctuation. The disturbance of banks is generally attributable to one of two causes:

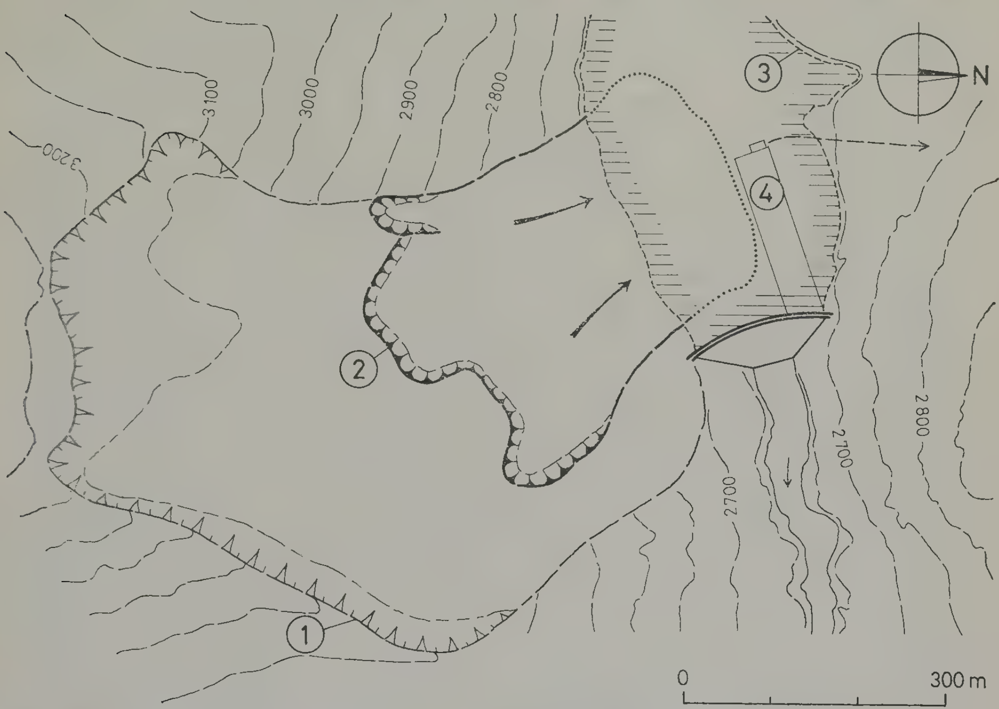


Fig. 10–11. Map showing a rockslide into the Tablachaca reservoir on the Mantaro river in Peru; the outlet installations are threatened by the slide; 1 — head scarp of the fossil rockslide, 2 — recent rockslide, 3 — Tablachaca reservoir, 4 — outlet installations (after Novosad, 1979).

(a) As a result of flooding, the stability conditions of some of the rocks are changed; if the rocks become saturated with water, their weight decreases due to buoyancy, although the loading above the water level remains unchanged. The cohesion of fine-grained soils may also be appreciably reduced by flooding. If there is an abrupt lowering of the water level the rock comes under pressure from water seeping back into the reservoir. All these factors may cause sliding of the reservoir banks.

(b) Wave action caused by wind and the wash of motorboats erodes the banks and in the same way as marine abrasion, creates a typical platform backed by a scarp. In contrast to natural lakes, reservoirs usually undergo large fluctuations of the water level depending on the demands of the power-plant. The fluctuations of the water level may be regular both on a daily and a seasonal cycle. Thus, the water-line sediments are eroded and the product of erosion falls down towards the bottom of the reservoir. The slopes are thus undercut and the stability of the banks is disturbed.

As an example of this phenomenon the landslides which occurred on the banks of the Columbia River may be cited. These greatly hindered the construction of the Grand Coulee Dam and increased considerably in frequency during the filling of the reservoir and in subsequent periods of drawdowns (Fig. 10–12). The Columbia River valley is carved in glaciofluvial and lacustrine sediments deposited in the Late Pleistocene glacial period. These deposits of sand, silt and gravel are preserved as terrace remnants on both sides of the valley. They have readily succumbed to sliding as a result of river erosion, and their susceptibility to failure increased further from the effects of the construction work. Sliding into the reservoir was intensively studied

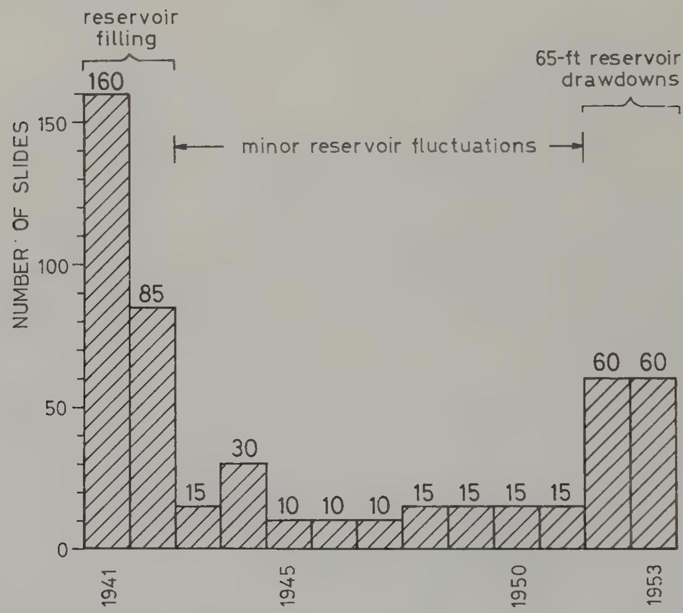


Fig. 10–12. Histogram showing the frequency of landslides in the reservoir of Grand Coulee Dam from 1941 to 1954. The total number of landslides was estimated at about 500 in this period (after Jones et al., 1961).

during the period 1942–1955 in order to establish a criterion for predicting which banks were most threatened by landslides. A detailed analysis of nearly 500 landslides, taking into account the type of material, the ground water conditions, the initial slopes of the banks and their submergence, made it possible to develop a method for predicting the stability of the natural slopes of the reservoir (Jones et al. 1961).

Landslides on the banks of reservoirs occur very frequently in the Apennines and their study is there a particularly important task of engineering-geological investigations. The flooding of the slopes and the fluctuations of the water level trigger off movements in temporarily stable slide areas, and the ensuing slope failures are a serious cause of reservoir silting (Segrè, 1924).

The question of the stability of mountain slopes flanking reservoirs became a subject of special interest after the disaster of the Vaiont Dam in the Italian Alps in 1963 (Fig. 10–13). The Vaiont Dam, one of the highest arch dams was erected in a deep narrow valley of a tributary of the river Piave, the slopes of which consist of interbedded Jurassic and Cretaceous limestones and marls. The construction work began in 1956 and was completed in 1960. In 1963 a huge rock mass, about 260 mil-

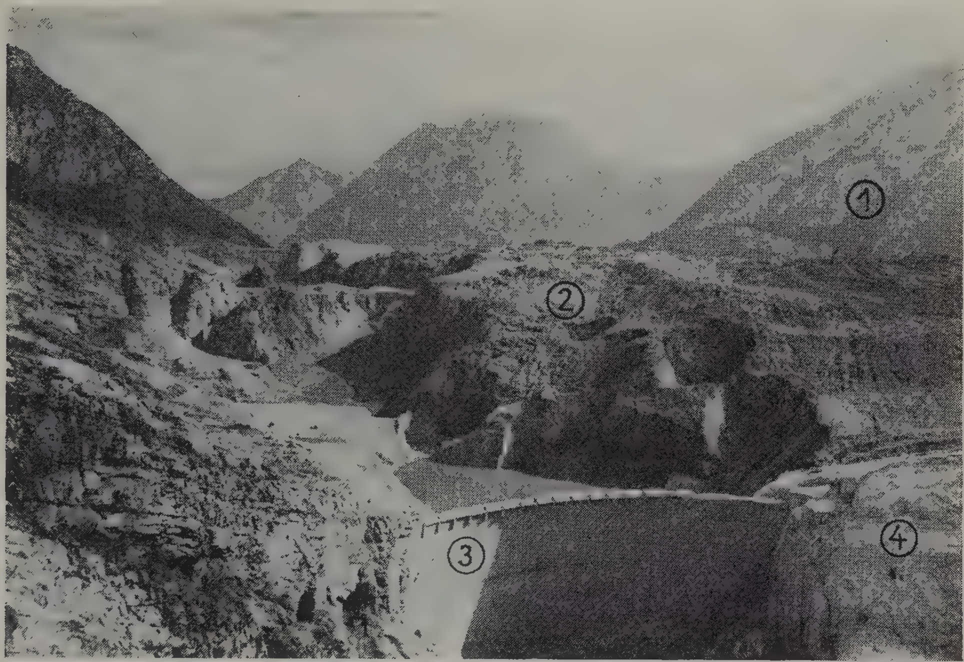


Fig. 10–13. Rockslide of Jurassic limestones in the Vaiont reservoir (from picture postcard). 1 — sliding surface, 2 — slipped limestones, 3 — arch dam, 4 — limestone laid bare by flood wave.

lion m^3 in volume, suddenly slipped from the slope of Monte Toc, forming the left-hand side of the valley, into the reservoir. The greatest damage was produced by a wave of water more than 100 m high which overflowed the dam, destroyed the town of Longarone, devastated the reservoir banks and the river valley for a long distance downstream (Fig. 10–14). Over 2000 people were killed. Despite this enormous damage the dam itself was only slightly impaired. The geological structure of the left slope which was not conducive to a situation of stability is shown in Fig. 10–15.

The cause of the disaster has been exhaustively studied and discussed (e. g. Selli, Trevisan et al. 1964, Müller 1964, Kiersch 1964, Mencl 1966, Stapledon 1976) and it appears that the last word on the cause of this catastrophic event has not yet been written.

The purport of the studies performed was that the geological investigations of the foundation conditions of the dam site were adequate to the importance of the structure, as can also be deduced from the fact that the dam sustained the enormous surcharge at a sudden overflow by a water wave one hundred metres high. Although the susceptibility of the left slope to sliding was recognized during the investigation works, it did not indicate that it might develop into an abrupt collapse of the slope.

Fig. 10–14. Map of the Vaiont rockslide in the Italian Alps; a — dam, b — outline of the head area, c — area devastated by the high wave of water and by air pressure, d — lakes, e — outline of the slide body (after Selli and Trevisan, 1964).

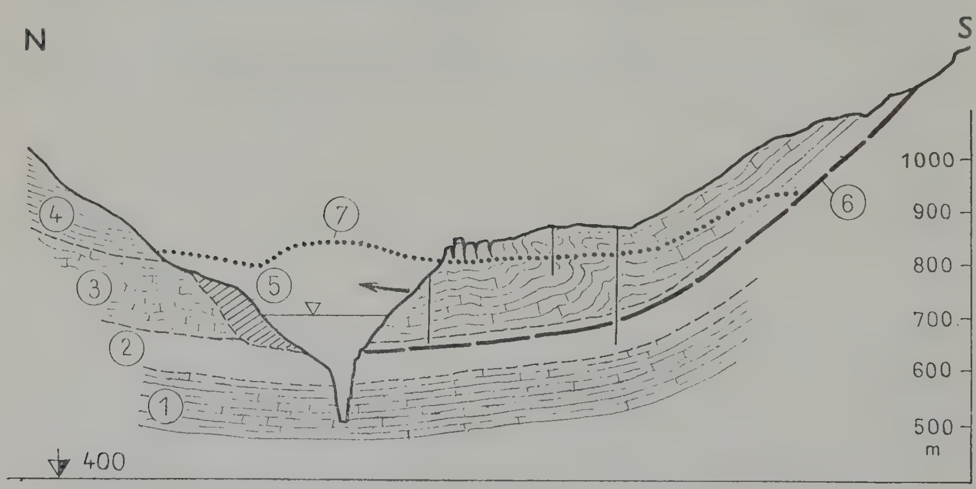


Fig. 10-15. Huge block of Jurassic and Cretaceous limestones slipped into the Vaiont reservoir, causing disastrous flood in the Piave-river valley (Selli, Trevisan et al. 1964); 1 — Dogger limene, 2 — thin-bedded limestone with clay interlayers (Malm), 3 — Cretaceous thick-bedded limestostone with chert, 4 — Cretaceous marly limestone, 5 — relic of an old landslide, 6 — slide surface, 7 — valley blocked by landslide material.

When the first slide occurred (volume of the slipped rock mass was 700,00 m^3) after the partial filling of the reservoir in 1960, a precise monitoring of the movement on the left slope was begun. The measurements showed that there was a slow creep of the rock mass, which character the motion had retained up to the very instant of failure, and there was no reason to presume such a sudden acceleration of the process (a creep rupture).

An example of detailed study of slope movements caused by water-level fluctuation in a reservoir is that undertaken at the Gepatch Dam in the Austrian Tyrol. After the reservoir was filled in 1965, some old movements were renewed on the left flank. The slope had undergone gravitational sliding in Postglacial time when about 100 million m^3 of rock moved and compressed glaciofluvial deposits on the valley bottom. Recent movements were measured systematically throughout the period 1966–1969 by means of a line of stakes. Measuring devices were also set up in several exploratory galleries driven into the slope. Figure 10–16 shows the results of the measurements in two profiles. The vertical component of the movement was the more predominant in the upper part of the disturbed slope, and the horizontal component was the greater at the foot of the slope. The vectors measured in the gallery showed that the front and distal ends of the gallery (in profile b) moved at approximately the same rate, indicating that the disturbance of the slope reached beyond the end of the gallery to

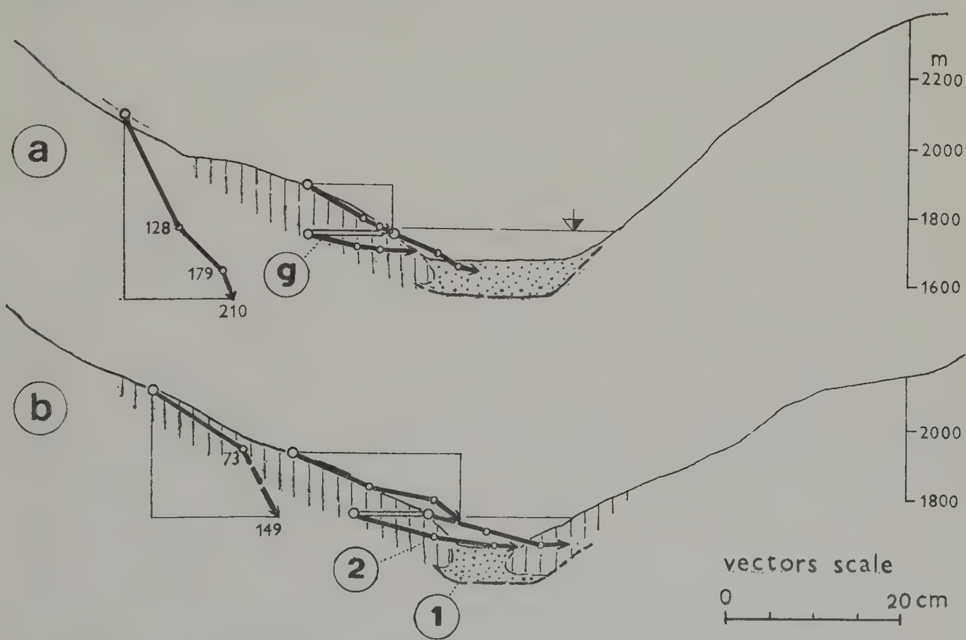


Fig. 10–16. Sections across the Gepatsch reservoir in the Tyrol; gravitational deformations on the northern slope were monitored from 1966 to 1969; 1 — stressed glaciofluvial sediments, 2 — gneiss and weathered rock disturbed by gravitational movement, g — galleries (after Neuhauser and Schober, 1970).

a distance of 185 m from the surface of the slope. The gallery in profile (a) was 244 m long; the rock was strongly shattered and loosened to a distance of 90 m. Open or loam-filled fissures were numerous and mylonitized zones were found 94–99 m and 227–234 m from the slope surface. Both of these shattered zones were almost parallel to the surface of the slope. The observations revealed a relationship between the slope movements and fluctuations in the water level, and their gradual decrease (Laufer et al. 1970, Neuhauser and Schober 1970).

These findings are relevant to the designing of reservoirs in the Flysch Carpathians, where there are many potential slide areas. Subsequent to the filling the reservoir near Roźnów on the river Dunajec in Poland, the flanks formed of weathered flysch material were strongly disturbed by slope movements, which extended high above the water level.

In considering alternative designs which differ with regard to the highest water level that is to be maintained in the reservoir, the final decision may rest on the cost of stabilizing the banks, especially where the slope movements might cause damage to valuable or built-up land.

A detailed investigation of the reservoir flanks was conducted for the Slapy Dam on the river Vltava in Bohemia. The banks are formed mainly of solid rock, except for a few sectors in which there are terrace deposits and fine sand covered with loess loam. The stability of the flanks was made evident in 1954, when flood water filled the reservoir almost to full height. As the construction of the control weir had not yet been completed, the water level rapidly dropped 17 m to the level of the crest of the dam. The drop in water level had no unfavourable effect on the rocky flanks and only in a few places did blocks slip along joint planes dipping down the slope; neither

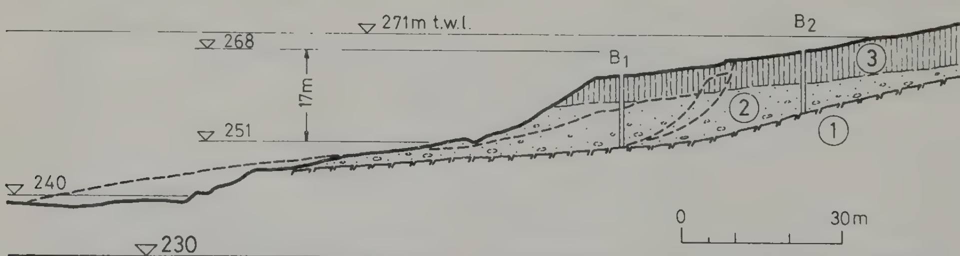


Fig. 10–17. Profile of the bank of the Slapy reservoir in Bohemia, disturbed by sliding after a 17 m-drawdown in 1954; 1 — amphibolite, 2 — terrace sand with gravel, 3 — loess loam.

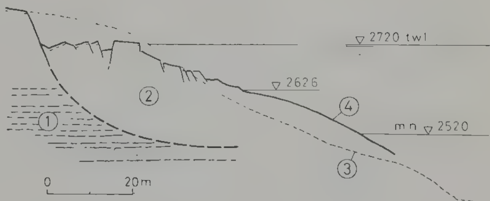
was there much effect on banks composed of terrace sand and gravel. As had been expected, the slopes of fine sand and loam were disturbed. Figure 10–17 shows such a damaged sector, where loess loam up to 7 m thick lies on fine sand covering the terrace gravel. Fine sand started to move under the pressure of water flowing back into the reservoir from the banks after the sudden drawdown. The loamy cover was disrupted into blocks that slid into the reservoir.

Fig. 10–18. Banks of the Nechranice reservoir damaged by sliding after lowering of the water level (photograph by Rybář).

In the Nechranice reservoir in north-western Bohemia, the flanks of which are formed of Neogene clays and silts, extensive sliding occurs whenever the water level is lowered and the newly formed cliffs cave headwards (Figs. 10–18, 10–19).

After the filling of the Nechranice reservoir, ancient landslides were also activated immediately below the dam as a result of water percolating from the reservoir through a permeable coal seam around the dam and then emerging downstream of the dam in several springs (total yield about 0.3 m^3 per minute). In order to stabilize the disturbed right-hand slope, a stabilizing gravel fill was constructed and the springs were captured and drained.

Fig. 10–19. Section of a landslide in the Nechranice reservoir after a 20 m-drawdown; 1 — Neogene clays with sand beds, 2 — slipped clays, 3 — original land surface, 4 — ground surface after drawdown.



Landslides and slope deformations provoked by fluctuation of the water level in new reservoirs have also been intensively studied in the Soviet Union and the results of geological investigations have been summarized in several monographs. These phenomena were described, for example, in the reservoirs on the river Angara (Odintsov, Palshin et al. 1963) and the river Kama (Pecherkin 1969). The role of the morphology and structural history of reservoir banks in determining their stability has been studied in detail by Zolotarev and Skvortsov (1961).

10.4 The effect of landslides on the permeability of the reservoir

Investigation of the permeability of the reservoir banks has in some cases revealed the existence of a side valley buried by an ancient landslide. If it were not found, considerable water losses might occur through the permeable slide debris and valley alluvium after the filling of the reservoir.

Just such a problem was encountered at the Senaiga dam site in the Italian Alps (Ciampi 1958). An arch dam 80 m high was built in a gorge cut in firm Lower Cretaceous limestone. The bottom and banks of the reservoir are formed of fairly impermeable rocks, except for a short sector on the left bank close to the dam site where the limestones are strongly disturbed. Exploration by means of boreholes and galleries showed that the rock was not in original position and had slipped down filling the valley and pushing the river over to the right bank, where a new channel was eroded. The limestones moved a relatively short distance so that their stratigraphic continuity was preserved; only in places did they show the character of rock debris. Further exploration revealed the precise form of the ancient valley and the nature of the slipped material.

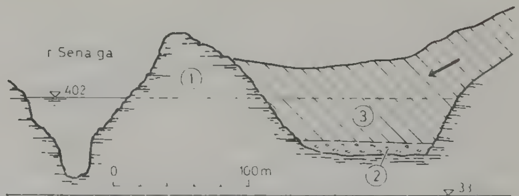


Fig. 10–21. Section of the dam site on the river Senaiga, including the old landslide-blocked valley; 1 — Lower Cretaceous limestones, 2 — glaciofluvial deposits, 3 — slipped limestones filling the old valley (Ciampi 1958).

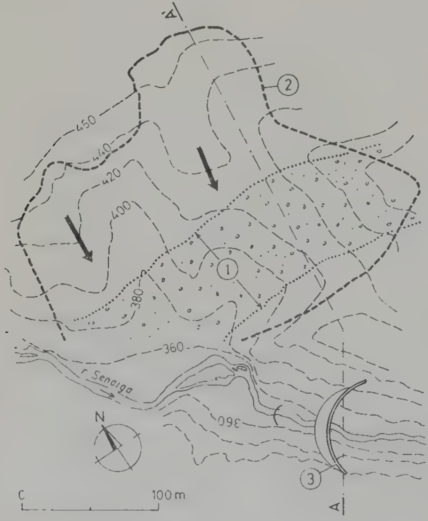


Fig. 10–20. Map of dam site on the river Senaiga; 1 — old valley, 2 — boundary of landslide area, 3 — dam (Ciampi 1958).

The floor of the old valley was covered with 5 – 6 m of very permeable glaciofluvial deposits with boulders. The slipped limestones were less permeable, because the fissures were filled with clayey material. Figure 10 – 20 shows the extent of the landslide and the approximate course of the ancient valley. From Fig. 10 – 21 it is apparent that the valley was blocked by a natural dam which at the level of impounded water (el. point 402), is immersed to a height of 44.50 m.

Permeability tests showed that not only the glaciofluvial deposits but also the weathered limestones along the old surface were highly pervious. Water losses through the slipped material were small, and it was not therefore necessary to seal the ancient valley up to its full height. In order to seal the glaciofluvial deposits three galleries were constructed one above the other, with interior diameters large enough for grouting units to be installed in them. The galleries were completed in 1954 and grouting was then carried out in two stages.

In the first stage, a grout curtain was formed in the limestone to a depth of 3 m below the lower gallery. The grouting holes which were 3 m long, spaced 1.7 m apart and arranged in two parallel rows, were filled with cement grout. After the first stage of grouting the reservoir was filled as a trial but when the water loss attained 2.1 m^3 per minute with the reservoir only partly filled, the water was emptied again.

In the second stage the grout curtain was sunk 25 m below the lower gallery. The grout holes were spaced 3 m apart and control holes were inclined at 30°. For the injections, cement grout was used, and a mixture of cement and fine sand was used in sectors of very high permeability. The average amount of dry mixture was 245 kg per 1 m of curtain hole length, and 83 kg per 1 m of control hole. The slipped limestones above were grouted from the upper gallery to a height of 10 m above its roof; the holes were inclined at 45° and spaced 3.5 m apart. Drilling in the slipped limestone was difficult since the walls of the holes were unstable, tending to collapse. In the limestone debris the consumption of grout was 35 kg per 1 m, whereas it amounted to 582 kg per 1 m in the holes made at the contact with the bedrock. Although these corrective measures were successful, they involved a considerable delay in the construction work and much extra expenditure.

The above examples show how slope movements may interfere with the construction of dams and their operation in a number of different ways. The problem of the stability of slopes both at the dam site and in the reservoir created by the dam should therefore be studied in all its aspects and in the greatest detail. Where areas susceptible to sliding are concerned, the geologist should provide the dam designer with the most comprehensive information possible, not only with respect to the properties of the rocks and their state of tectonic disturbance but also with respect to the geological history of the valley, the age and depth of the slides and the characteristics of the landslide material.

In an adequate engineering-geological report the designer has a powerful means of arriving at criteria on the basis of which decisions concerning the location, type and size of the structure may be taken. In complicated natural conditions, however, the available investigation methods cannot guarantee that no problems will arise for the builders, since the intervals between individual boreholes and investigated sections remain unexplored and the intervening geological conditions have to be interpolated. It is therefore essential that the engineering geologist should be on hand during the construction work and after the dam has been put into operation in order to check the results of the investigations with the real situation and warn of any dangers that the results may predict.

Chapter 11

LANDSLIDES AND ROAD CONSTRUCTION

The chief job of the designer and engineering geologist is to prevent potential slope failures. In this respect, road construction presents several different kinds of problems, such as selecting the best route (section 11.1), deciding on the depth of cuttings, designing the slopes, the drainage systems and the retaining structures of the cuttings (section 11.2), designing the shapes of embankments (section 11.3), and if necessary, preventing slope movements which might threaten tunnel construction (section 11.4).

It was during the construction of the early railways that the importance of engineering-geological studies was first recognized. This was followed by activities of the Swedish Geotechnical Commission (established in 1914), whose findings led the way to the development of the science of soil mechanics. Since that time field and laboratory investigations have become an every-day part of the design process.

In the author's experience the development of soil and rock mechanics has brought with it two dangers The first of these is that the problems of slopes may be considered from the viewpoint of mechanics alone, without reference to the many engineering-geological factors that may be involved in the susceptibility of the slope to failure. Secondly, the adoption of several numerical techniques referred to in textbooks may inspire confidence in the absolute validity of their results, which are in fact subject to the limitations inherent in the techniques themselves. It has been the object of many present-day workers to convince the engineering geologists and designers that the problems of rock and soil mechanics cannot be solved only on the level of mechanics handbooks, however elaborate the computation procedures may be.

11.1 Selection of the route

With the exception of mountain areas, the route is dictated mostly by considerations other than purely engineering-geological ones, especially under the conditions of the densely populated areas of Central Europe. Therefore, the part played by engineering geologists tends to be a secondary one involving the laying out of cuttings, embankments and bridges from the point of view of the stability of the foundation.

Before the design work is begun, it is necessary to decide whether the excavated material from cuttings will be suitable for constructing embankments. Modern heavy dynamic rollers are capable of compacting earthen materials, even those of a clayey character such as silty claystones. This cannot be done, however, with plastic, saturated Tertiary clays. Fortunately a considerable area of Central Europe is covered with loess or loess-loam, which is a suitable material for filling if heavily compacted. Thus it is the task of the geologist to insist that the designer locates the sub-grade level in cuttings at some point above the bottom of the loess cover. This of course means that embankments will tend to be built higher. Even though they are more expensive, high embankments can be made safer than cuttings with high slopes. If necessary, bridges of precast construction may be used in place of embankments. Another reason for minimizing excavations in clay is the need to cover the excavated clayey slopes with sand and gravel, which are often of rare occurrence in Tertiary basins.

The preference for embankments over cuttings also appears in other difficult situations. In this respect we may recall the statement of Terzaghi and Peck (1948): "Experienced engineers always locate new lines of transportation with a view to avoiding cuttings in troublesome ground as far as conditions permit."

Even in hard crystalline rocks this observation holds true; in the interest of safety slopes in crystalline rocks always tend to be made less steep than allowed by the design. This adds to the amount of expensive rock excavation that is carried out and increases the problem of where to deposit the surplus material. On the other hand, shallow cuttings can easily be excavated in the weathered surface of the rock, and the balance of material needed for embankments can be obtained from pits opened in the weathered rock surface. A better solution still is to provide more embankment material by making the cuttings wider. Moreover, with widened cuttings treatment of the rock walls can be neglected, making for an important saving in costs and manual work, and an increased degree of safety to traffic (Fig. 11 – 1).

The slopes of hills allow a lot of scope in the way that the road is landscaped. It is first of all necessary to obtain a general idea of the depth of the stable bedrock at

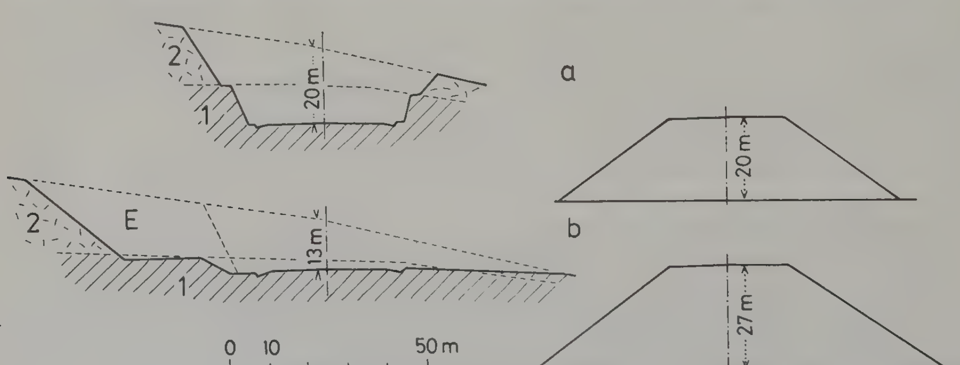


Fig. 11–1. Two alternative levels for an express highway in hard crystalline rocks (1), which are weathered near the ground surface (2); in (a) the cutting is deeper and the embankment is lower than the counterparts in (b), where the excavation is largely confined to the weathered rock; E — borrow pit created by enlargement of the cutting.

different levels on the slopes of the area, as far as any generalization is possible. The conditions of the Carpathian flysch areas will serve as an example. Fig. 11–2 shows the different ways in which a natural bench on a valley slope might be interpreted. In profile (a) the accumulation of colluvial material may or may not be sufficiently

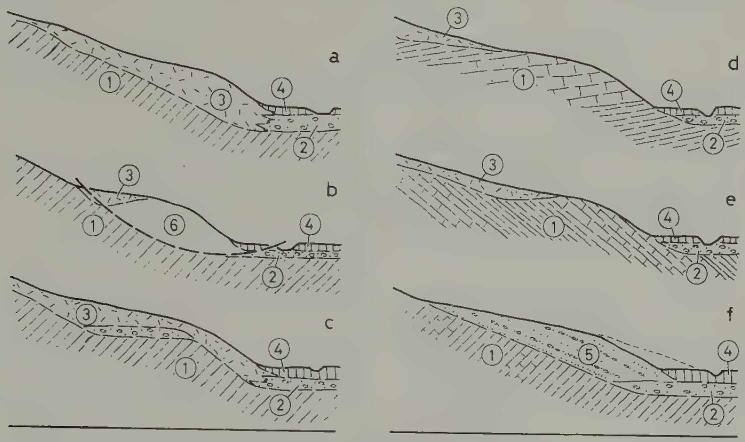


Fig. 11–2. Possible geomorphological interpretations of a natural bench on a slope in the Carpathian flysch area; (a) bench formed by the accumulation of colluvial deposits (3); (b) by slope failure, (c) as a river terrace, (d) and (e) by selective weathering, (f) as an alluvial fan (5) shaped by lateral erosion; 1 — bedrock, 2 — terrace gravel, 4 — Holocene deposits.

stable for the location of the route; the stability will depend on ground-water conditions. A careful investigation is also necessary in interpretation (b), since the platform has been shaped by a landslide. A borehole located above the toe will usually indicate whether there is alluvial gravel and test the landslide hypothesis. Also an investigation of the area of the potential head scarp may be useful. The platform shown in (c) is a stream terrace, and offers a good siting for a roadway. Similarly, the platforms formed by a selective denudation in situations (d) and (e) are also suitable sites. A flat alluvial fan as in (f), formed by lateral erosion, has often turned out to be treacherous if it has been cut into by excavation. This instability is mostly caused by

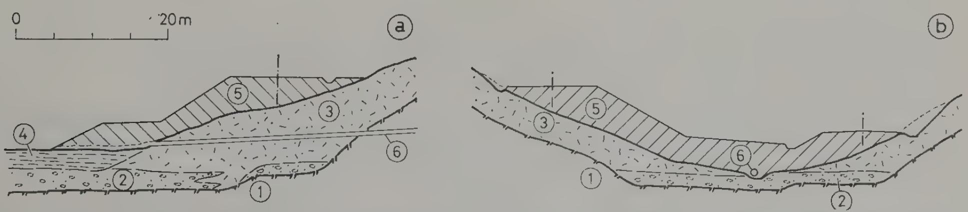


Fig. 11–3. The construction of a road on the toe of a slope in Palaeogene Flysch; (a) the road is situated on embankments built partly on the valley floor. If possible, side valleys are incorporated in the construction of the roadway (b); 1 — bedrock, 2 — sandy gravel, 3 — colluvial loam, 4 — Holocene loam, 5 — embankment, 6 — drainage boreholes.

the presence of water and of thin layers of clayey soils deposited within the otherwise coarse material of the fan.

When taking the route further up the hill, the thickness of the talus material becomes important. If the thick mass of accumulated talus material is not stable, then the construction of an embankment may be a reasonable solution (Fig. 11–3a). Further uphill, there is a considerable thickness of talus material and a simple solution (Fig. 4a) may not be possible. A side valley would make it possible to take the ascending route away from the talus (Fig. 11–3b). Or, if necessary, a retaining wall (11–4b) may be constructed to support the fill, using piles and anchors. When nearing the tops of the slopes the rock is generally at a reasonable depth and hillside cuttings are then possible (Fig. 11–4c).

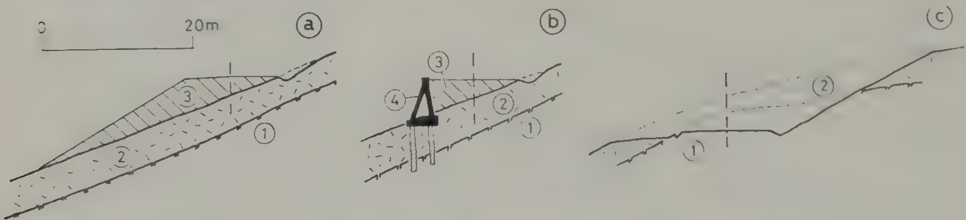


Fig. 11–4. Siting the route mid-way up the slope (a) may not be possible for lack of stability. If building round a side valley (Fig. 11—3b) is not possible, retaining walls or other artificial structures are necessary. On the tops of slopes (c) the thickness of the colluvial loam is small and the excavation of a cutting may be safe; 1 — bedrock, 2 — colluvial loam, 3 — fill, 4 — retaining wall.

Hand in hand with the above considerations, hydrological conditions must also be taken into account. Several specific aspects of hydrogeology will be dealt with in the next two sections, but one point of primary importance to be considered first of all, is that the road level should never be located within the zone of active uplift of the ground-water. The latter occurs at the bottoms of flat depressions filled with impermeable rocks or soils of Cretaceous, Tertiary or Quaternary age, overlying older and more permeable rocks (Fig. 11–5). The uplift force is capable of weakening the soil to such a degree that even shallow excavations for ditches give rise to sliding move-

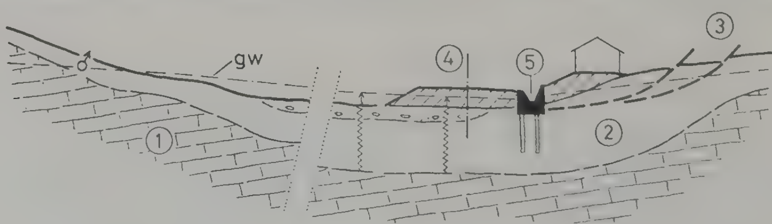


Fig. 11–5. Diagramatic profile showing conditions in depressions filled with impermeable rocks (2) overlying permeable water-bearing rocks (1). When the road is located on the valley floor, even small excavations generate landslide. 3 — recent landslides, 4 — road embankment, 5 — retaining wall founded on piles.

ments. Lowering the ground-water table by means of vertical boreholes may not be effective, because the water in the bedrock is supplied from a very wide area round about. Even the foundation excavations may cause slope movements, and the bearing capacity of this type of ground is not sufficient for embankments. The best solution is to relocate the road on the slopes of the valley where drainage is possible.

Where possible, wet hill slopes should be avoided when siting a roadway, and the presence of springs is also a disadvantage (Fig. 11–6). Slopes facing south are to be preferred since they are covered with snow for a shorter time.

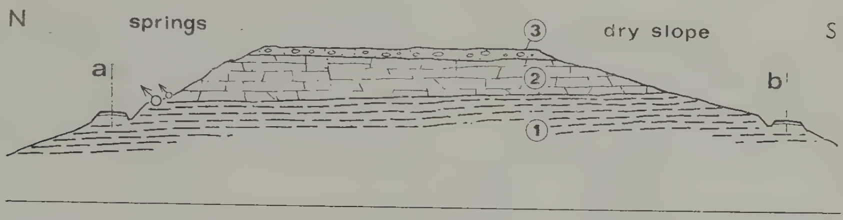


Fig. 11–6. The orientation of impermeable strata may cause a slope (a) to be wetter than that (b) on the other side of the hill. Moreover, a slope exposed to the north is less suitable for the location of a roadway than a south-facing slope, 1 — Cretaceous marl, 2 — sandstone, 3 — sandy gravel.

The outcrops of beds in the slopes of broad valleys often exhibit an upheaval of several metres of height caused by the release of overburden pressure on clayey soils or weak rocks as the valley is eroded. If the clayey layers alternate with more permeable strata, the discharge of ground-water does not occur so much on the slopes of the main valleys, but rather in the secondary valleys inclined in a direction at right angles to that of the principal valley slopes (Fig. 11–7).

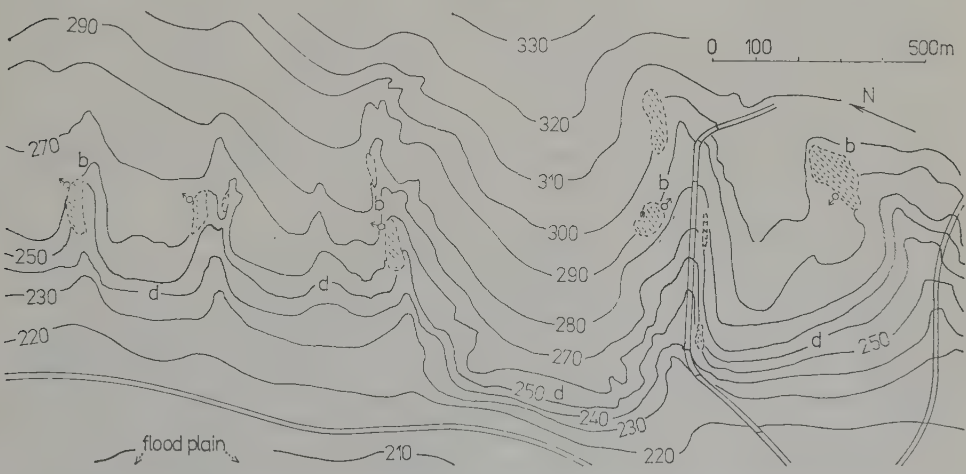


Fig. 11–7. A slight rebound upheaval at the margin of a hilly ground formed of clayey rocks may make the slopes (d) drier. Ground-water flows out as springs in secondary valleys (b).

11.2 Preliminary work on the site

Long before the construction of the roadway begins, the site must be drained. Significant lowering of the ground-water table by subsurface drainage may take many months. In the areas of the Carpathian flysch rocks it usually takes a period covering one winter before any considerable decrease appears in the discharge from deep drainage works. If a decrease does not occur, several supplementary drains may be necessary, and these should, of course, be active prior to the commencement of construction work.

The use of drainage trenches in road construction is subject to the same conditions as subsurface drainage. Moreover, drainage trenches often serve as the outlets for drainage boreholes. The best plan, therefore, is to construct drainage trenches soon after the drainage boreholes have been completed, in the autumn preceding the first year of construction work, after harvest.

It is important to note that subsurface drainage must not be applied to conduct surface water.

11.3 Cuttings

The design and construction of cuttings is a difficult task, and in spite of great dilligence on the part of the designers, failures are not rare. It is proof of how complicated the problems can be; the problems arise from several sources, as follows:

– A number of geological factors may not be taken into account. For example, the influence of high initial stress in the ground is not widely recognized, or it may happen that the presence of separation planes is not discovered in the investigation.

– The assessment of ground-water conditions may be unsatisfactory. Very often the most permeable layer within the slope has not been properly defined, so that the actual potential slip surface has not been considered. Alternatively, in the case of clayey soils, information on the pore water pressure may be lacking. During the excavation work, especially when this is carried out in a dry summer, it may happen that spots of wet material escape attention.

– There may be a mistaken belief that a single method of stability analysis exists which is capable of giving a safe solution to the slope problems in hand. The more clayey the earthen material, the more intricate the approach needs to be.

– There may also be too much confidence that "cosmetic" measures (surface draining, a thicker sodded layer) are capable of significantly increasing the stability of the slope.

As mentioned earlier, the engineering geologist often only has room to manoeuvre in the vertical plane as far as location of the road is concerned, and full use must be made of this in cuttings. Thus, for example, considerable areas of Central Europe are covered by thick loess layers overlying clayey soils or weak rocks. Water penetrating the loess collects on the surface of the bedrock and this is commonly the cause of slope failure. It is generally recommended that the subgrade be raised at least one metre above the bedrock surface. In this way frost damage to the clayey bedrock can also be avoided.

Lowering the level of the roadway may sometimes have a favourable effect (Fig. 11–8), if the subgrade can be located, e. g., in the gravel terrace which then drains the cutting.

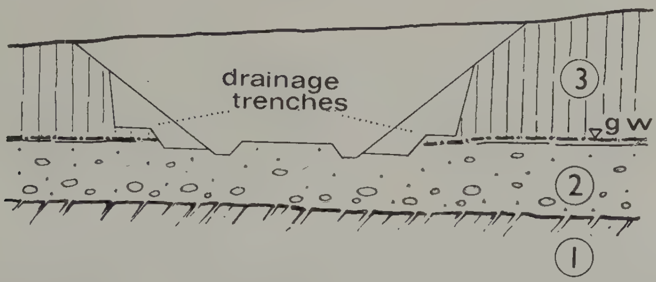


Fig. 11–8. A deep cutting may reach down to the saturated terrace gravel; the stability of the slopes is provided by drains; 1 — bedrock, 2 — sandy gravel, 3 — loess loam.

The need to drain the slopes and basements of cuttings by means of horizontal boreholes may be another reason for raising the level of the road. It is best to drain the ground before excavation work begins, and this is possible by boring the drainage holes from the ends of the cutting, beginning several metres below the level of the road subgrade. Since a reasonable length for the boreholes is about 150 m, a cutting 300 m long can be started in this way. This length gives the impulse to the consideration of the level of the line.

The question of the shape and gradient of the slopes of cuttings has been dealt with in Chapters 4 and 7. In the following the problems are summarized from the point of view of engineering practice.

11.3.1 Deep failure of slopes in clayey rocks

The various causes of deep failure in slopes of stiff clayey soils and weak rocks can be illustrated in terms of the overall safety factors:

For a dry slope of gradient 1 : 3 and height 10 m, the general safety factor will be 2.8 to 3 according to the classical solutions, or 2.85 according to the FEM analysis, taking the coefficient of lateral stress as 1.

With a slope height of 13.5 m, the classical solution yields a safety factor of 2.4 to 2.6; according to FEM values for the safety factor are 2.4 and 1.44, when the coefficient of lateral stress is 0.75 and 1.5, respectively.

A dry slope 16 m high exhibits a safety factor of 1.9 to 2.1 according to the classical solutions, but only 1.8 and 1.54 according to FEM, when the coefficient of lateral stress is 0.75 and 1, respectively. A coefficient of 1.5 does not yield a stable state. The above values of 1.8 and 1.54 increase to 2.2 and 1.67 if a protective fill 1.5 m thick and 6 m high is considered.

With the ground-water table at the mid-height of the slope, the 16 m slope displays a safety factor of 0.98 by FEM, but the latter value increases to 1.67 if a protective fill 1.7 m thick and 6 m high is included.

From the foregoing several rules emerge: With values of the coefficient of lateral stress up to $K_0 = 1$, the results of the classical methods may be adopted, using Skempton's reduction coefficients R for the safety factors. The latter coefficients are of the order of 1.15 for $K_0 = 0.75$, and 1.35 for $K_0 = 1$. Nevertheless, if more accurate values are required, the FEM solutions for several typical cases can be used as a basis for the analysis of the new situation. The importance of estimating the value of K_0 cannot be overemphasized.

The decrease in the safety factor caused by the presence of ground-water can be expected to be greater than that indicated by the classical solutions.

A protective fill on the slope surface does not contribute much to the structural stability if the slope is dry. Conversely, the effect is very considerable for wet slopes, much greater than the classical solutions are likely to indicate. Few slopes, however, are fully drained, and taking into account its value in reducing frost damage (see the following) the protective fill must be considered an indispensable tool in the engineering of cuttings.

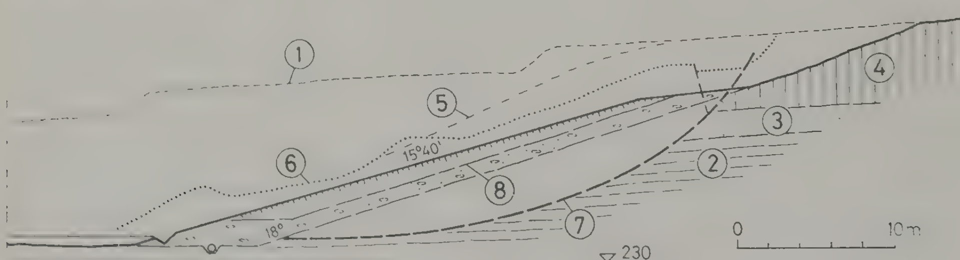


Fig. 11–9. Stabilization of a sliding slope by flattening it and protecting it with a fill; 1 — original ground surface, 2 — stiff fissured clay, Miocene, 3 — terrace sand, 4 — loess loam, 5 — original design of the cutting, 6 — profile of failure during excavation, 7 — assumed slip surface, 8 — protective layer, constructed in a sandwich form (70 cm gravel covered with loess and topped with a sodded layer).

In the authors' experience, slopes in stiff clays, gradient 1 : 3, have proved to be stable up to a height of 18 m if covered by a protective fill (Fig. 8–5). Slopes lower than 10 m may have a steeper gradient of about 1 : 2.5. If water is present in the slope, it should be drained. Stiff clays are mostly fissured and therefore permeable, too.

If former slip surfaces or faults are present in the ground, flatter slopes and protective fills of greater thickness are necessary. A cutting of the express-way near Brno in stiff Neogene clay is shown in Fig. 11–9. A section of the cutting 12 to 15 m high and 80 m long slipped during the excavation work, the angle of inclination of the unfinished slope being 18° when it collapsed. Owing to the presence of the slip surface the second stage of the excavation, which included the construction of a 1.8 m thick protection fill, was accompanied by small deformations. The slope was probably in a state of limit equilibrium, and thus it was possible to determine the angle of residual shear resistance (11°20′) for the clay (Lower Tortonian, $w = 25\%$, $w_p = 26\%$, $w_L = 69\%$, 42% minus 2 μm, illitic). Only the third and final design proved satisfactory.

Slopes constructed under similar conditions in the fissile Eocene claystones of the Central Carpathian Flysch (which is less tectonized than the rocks of the Outer Flysch) proved to be satisfactory if excavated with a gradient of 1 : 2 in the sound rock, and 1 : 2.5 in the weathered layers underlying the higher gravel terraces. Of course, gravel protection fills were applied.

When large horizontal stresses are present in the ground, protective fills are not capable of preventing slope failure. Under these conditions pile walls or invert frames (Fig. 4—12) are necessary to support the toe of the slope, where they have to stand up against a considerable force (section 8.5).

Where slopes higher than about 18 m are to be created the angle of inclination of the slopes must be decreased. An example was presented in section 7.7 of a slope 95 m high that remained stable for many years at an angle of 12.8° (gradient 1 : 4.45). The slope in this case was fully drained. More of the problems of high slopes will be said in Chapter 13.

11.3.2 The influence of water and frost

Generally they cause shallow landslides in otherwise stable slopes, even if the area is drained. In frosty conditions water enters the soil from strata within the slope, and the soil disintegrates rapidly during the subsequent thaw. Three days of sudden frosts of 12° to 15 °C were sufficient to cause disintegration even of the low slopes on

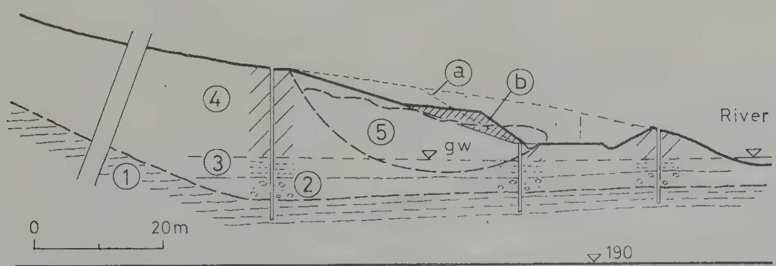


Fig. 11–10. Geological section of a railway cutting near Bílina in N. Bohemia; 1 — Cretaceous marls, 2 — sandy gravel, 3 — alluvium, 4 — ancient landslide, 5 — new slide (1966), a — original excavation, b — buttress of crushed basalt (Záruba and Šimek, 1970).

both sides of the excavation, and experience of this phenomenon has taught the designers of roads in the weak Tertiary rocks of Czechoslovakia that slopes not protected with sand or gravel will collapse sooner or later. Slopes covered with a protective layer may remain stable at an angle of 24° (gradient 1 : 2.2), provided that they are not high enough to be jeopardized by deep failure (section 11.3.1). A height of 7 m seems to be the limit for slopes in stiff clays, or 12 m for slopes in the fissile claystones of the Carpathian Flysch.

Loose clayey colluvial masses signal two dangers for the builders of cutting, one being the possibility of a deep movement owing to the unloading of material at the bottom of the mass, the other being sheet slides involving the surface layers. A careful investigation is necessary in order to obtain a reliable picture of how the slip surface is likely to develop. An example is shown in Fig. 11–10.

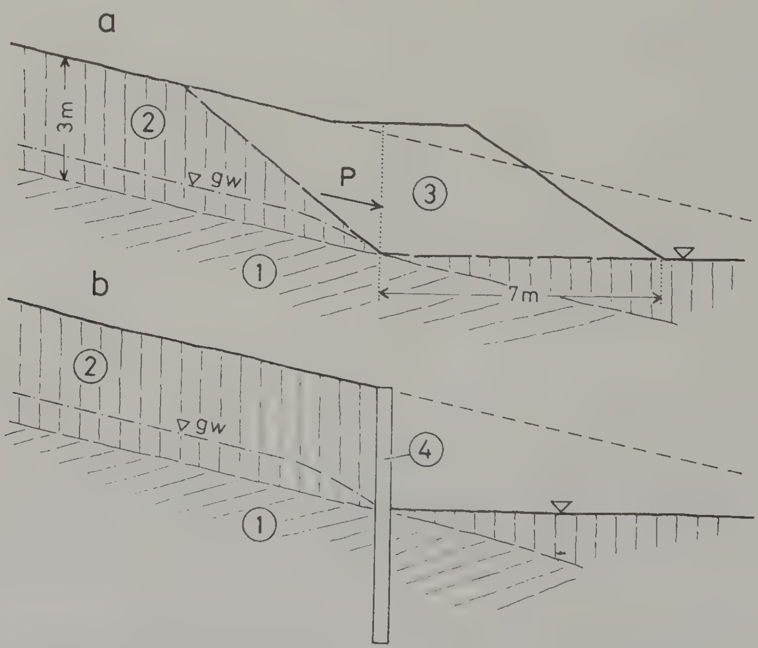


Fig. 11–11. Diagram showing the supporting measures that can be taken at the toe of an unstable clayey colluvial sheet (2). (a) The width of the fill (3) necessary to resist force *P* is excessive. (b) The supporting pile wall (4) reduces the amount of excavation required; 1 — Neogene clay.

The supporting fill necessary to resist the pressure of a few metres of loose clayey colluvial sheet, should be thicker than shown in the example of Fig. 11–9. In the latter case the force was of the order of some tens of kN. A force of 80 kN must be mobilized in the example illustrated in Fig. 11–11, and the dimensions of the supporting fill needed to fulfil this purpose are too great. In this type of situation pile walls are often preferred.

(a) When joints, bedding planes, schistosity planes and faults dip downslope, they often give rise to slides. An example of such slides is shown in Fig. 5–57. Closed joints free of clayey infillings may often allow a high degree of stability in hard rock slopes even when dipping at an angle of 35°, but a thorough investigation of physical processes developing along the joints near the slope surface is necessary (Sowers and Carter 1979). An angle of dip of 23° corresponds with the experience gained with continuous but closed joints, corresponding to the schistosity of amphibolite, if clayey coatings were present. Angles of 21° for the Palaeogene strata and 18° for Cretaceous fissile shales correspond with the respective states of limit equilibrium under the conditions prevailing in the West Carpathians. However, the slope movement occurs even when the beds are dipping at an angle of 11°, if the slope is flat and the colluvial cover is thin, so that surface water can easily penetrate into the rock along the steep transverse joints. In this case the slope failure spreads very rapidly and far from the cutting even when only few metres of it were excavated.

When important facilities are present near the edge of the proposed cutting, rock anchors can be applied to reduce the extent of the excavation (Fig. 8–20).

An example of movement along bedding planes of a cutting at Bohdalec, Prague is shown in Fig. 11–12. The cutting was intended to replace an old tunnel. The slope is formed of Ordovician shales with quartzite intercalations; the beds are openly

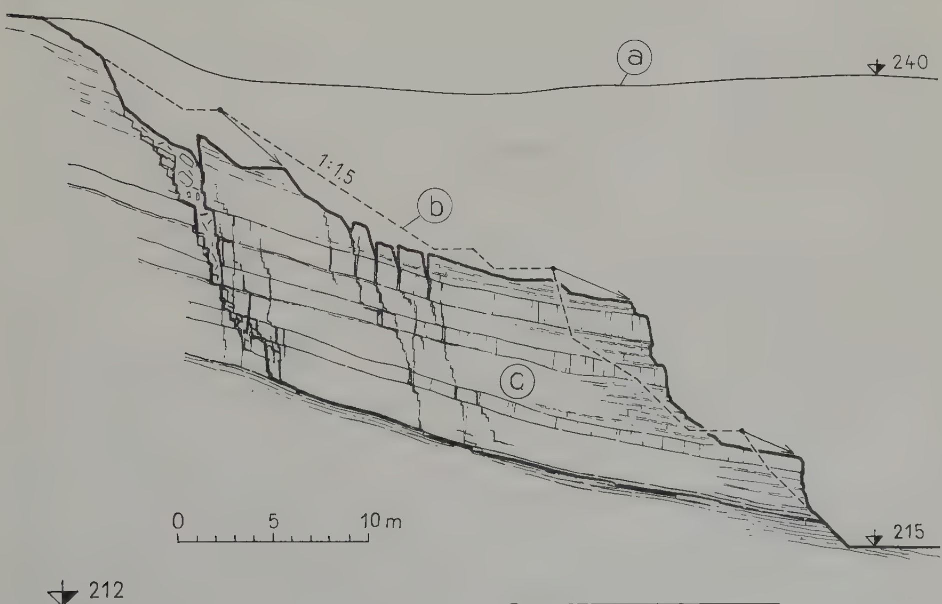


Fig. 11–12. Section through landslide in the cutting at Bohdalec in Prague; a — original surface, b — graded slope of the excavation, c — slippage of shales along bedding planes.

folded and in the south-eastern sector are inclined at 15 – 20° into the cutting. The rocks were set in motion during the excavation work in which the slope was graded at 1 : 1 in the lower part and 1 : 1.5 in the upper part, with three horizontal benches. In the winter, a block of about 8000 m^3 volume slipped into the cutting and a steep fissure developed. The landslide was confined to that part of the slope where shales formed a slight synclinal bend. The rock moved as a uniform complex, but areas loosened by the movement were traversed by numerous, approximately vertical fissures. In particular, the firm quartzite beds were cut transversely into small cubes surrounded by open cracks. In view of the gentle dip of the beds and the nature of the rocks into which there was no inflow of water, the movement was unexpected and difficult to explain. Forty years later, after many field tests have been carried out on the shales, it was established that shear strength parameters for the partly weathered shales sheared along the bedding planes were 0.07 MPa and 21°. Probably the release of stress brought about by the construction of the tunnel gave rise to a progressive development of local failures in the rock mass.

(b) Slope failures across the surfaces of weakening in hard rocks are difficult to analyse statically because the failures are preceded by the process of deformation which changes the interplay of forces. The problem is therefore statically indeterminate and the deformation methods of analysis are required in order to obtain a reliable picture of the behaviour of the slope. Field measurements made before and during the excavation work may provide a good alternative to static analysis, but perhaps a greater number of situations and models needs to be investigated and statically analyzed so that the simplified static computations can be carried out. In this respect a purpose-made classification of possible patterns of failure would probably be useful (Müller 1967).

An attempt to analyze the interplay of forces in a slope in order to obtain an approximate indication of its safety is shown in Fig. 11 – 13. A cutting for a road requires the excavation of a slope 15 m high in Lower Cretaceous rocks with flysch structure. The bedding is almost regular, dipping at about 10° towards the toe of the slope, and the presence of a set of transverse fissures does not suggest the existence of a continuous plane. Nevertheless, the slope fails along the surface *abcd*. According to Müller (1967) and Vardar (1974) a small deformation of the slope accompanying the excavation may produce a decrease in the horizontal stresses and a loosening of the rock mass between *b* and *c*. This is also suggested by the vector of displacements computed by the FEM. The computation takes into account the removal of the last 10% of the weight of the excavated rock. Owing to the decrease in the stress and the loosening that occurs, the mechanism of the rock body is not unlike that of a person lounging half-sitting, half-lying stiffly on a chair, and is statically represented by forces P_1 and P_2 in Fig. 11 – 13. Provided that the weight of the body can be resolved into reactions P_1 and P_2 which make admissible angles with the normals to the respective sectors of the slip surface *abcd*, and provided that the points of application of the above forces are situated within the central ranges of the sectors *ab* and *cd*, respectively, of the slip surface, a state of equilibrium is probably possible. The maximum values of the respective angles, 18° and 28°, were adopted from Table 3–1 by Záruba and Mencl (1976). The cohesion intercept of the shear strength along

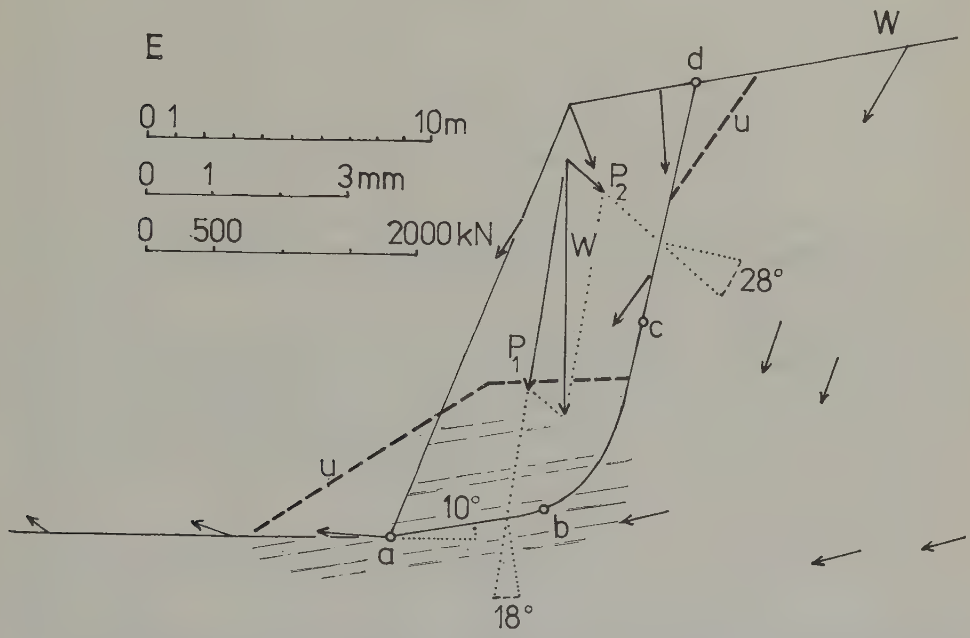


Fig. 11–13. Analysis of forces involved in the collapse of a steep rock slope during the excavations for a road near Hodslavice (Moravia). The rocks are sandstones and shales, Cretaceous Flysch; the collapse occurred along a bedding plane dipping at 10°. The displacement vectors analysed for the last 5% of the weight of excavated rock indicated that the rock could have been loosened in the sector *b c* of the developing failure surface.

section cd was ignored as already suggested by Terzaghi (1962). As can be seen, forces P_1 and P_2 only narrowly fulfil the above requirements for equilibrium. Clearly, more research is necessary before reasonable predictions of the behaviour of high rock slopes of this kind can be made.

Flattening of the slope will increase its stability, but a small flattening would probably not be very effective. The parallelogram of forces shown in Fig. 11–13 indicates that a horizontal force of about 350 kN is involved. A considerable part of the magnitude of this force may be resisted, for example, by making a bench fill of about 25 m^3 at the toe of the slope. The most effective solution would mean excavating the top of the slope and using the excavated material to make the bench (u). This could be done in the situation depicted in Fig. 11–13, the collapsed rock being used to form the bench. Such a solution has an additional advantage in that the bench serves as a catch area for falling rocks. However it is not good practice to design a slope which will fail and then depend upon support from a bench. In many cases there may be no room for a bench, and the only solution then is to anchor the slope.

11.4 Embankments

Weak subsoils under embankments can give rise to failures. The purpose of this section is to deal with landslides generated by the construction of road embankments. In this respect the following geological situations can be distinguished:

11.4.1 Weakness of the subsoil due to poor consistency

This is often the case with Tertiary and Cretaceous clays. Their stiff consistency enables embankments to be constructed up to a height of about 8 m without any special measures other than surface drainage at the top of the subsoil. Deep drainage (sand piles) and/or stabilizing berms are necessary for higher embankments. Sand piles not only provide for an accelerated consolidation of the soil, but also have the

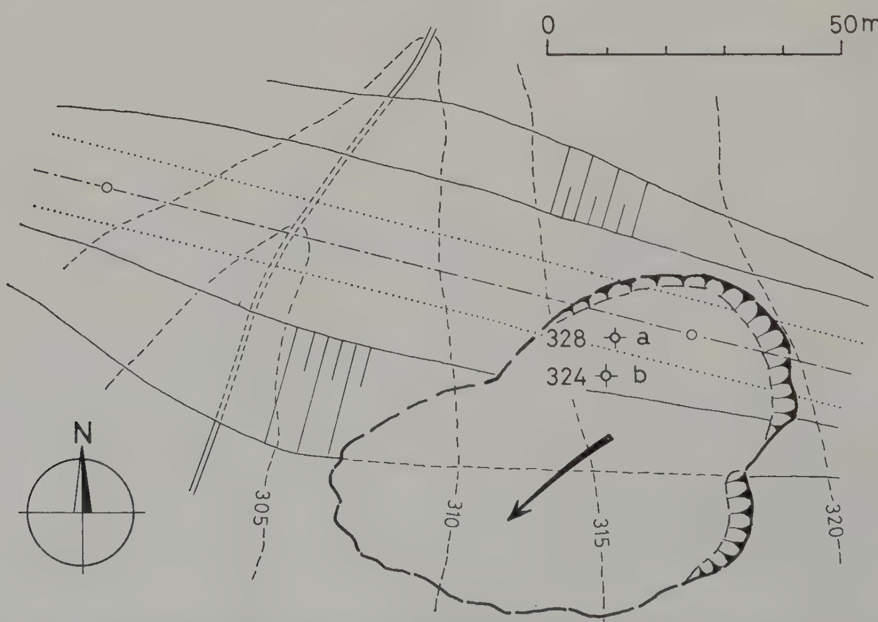


Fig. 11–14. Sketch map showing how embankment failure may be directed obliquely to the road axis if the ground surface is sloping. The designed height of the embankment in this case was 11 m, but the subsoil (Cretaceous marl) failed at a height of 7 m. Deep drainage ribs and a stabilization berm had to be constructed before the embankment could be completed; a — designed embankment surface, b — height of the embankment when failure occurred.

advantage that the total settlement of the subsoil is about two thirds of that accompanying a slow consolidation.

The inexperienced designer may well be advised here to make a study of profiles, not only in a direction normal to the axis of the embankment, but also in an oblique direction with respect to the embankment (Fig. 11–14).

11.4.2 Weakness of the subsoil due to the presence of water in thin sandy laminae within a clayey mass

This is often the situation in Neogene clays. Water in the sand layers may be under pressure owing to the slope of the ground, and the pressure increases under the weight of the embankment and the stabilizing berm. The greater the amount of material deposited on the slope, the greater are the uplift forces that result. What is more, the process of consolidation of the clay layers is slowed down. Consolidation piles and/or broad stabilizing berms are used where the ground surface is horizontal. The failure of the railway embankment at Podlešín near Slaný (Fig. 11–15) (Záruba, 1927)

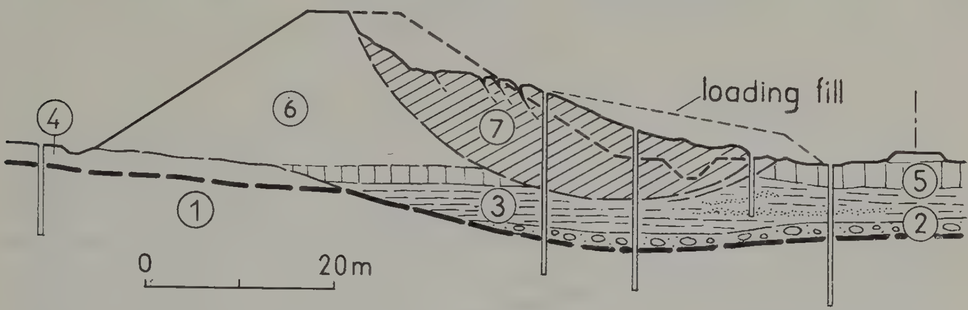


Fig. 11–15. Collapse of a railway embankment near Podlešín, Bohemia; 1 — argillites and sandstones, Permian, 2 — sandy gravel, 3 — clayey-sandy alluvium, 4 — loess loam, 5 — alluvial loam, 6 — embankment, 7 — slipped mass.

may be cited as an example. The 16 m high embankment was founded partly on solid argillites and sandstones of Permian age, and partly on clayey-sandy alluvia, overlying water-logged sandy gravels. In several test borings it was found that the water in the gravel was under pressure. The substratum was squeezed out progressively after periods of heavy rainfall, the material being heaved up into several bulges which blocked the channel of the nearby brook. The water thus impounded by the blockage threatened a building and road in the valley. Eventually the embankment was secured by a loading fill placed at the toe of the embankment.

Where the general ground surface is sloping, a solution can be achieved with horizontal boreholes by means of which the ground-water table may possibly be lowered to a level below the ground surface. Unhappily, the experiences with horizontal boreholes in this kind of subsoil have not been favourable, as already noted in section 8.3.

An example of an unsuccessful scheme is shown in Fig. 11–16. A road embankment up to 12 m in height failed, and its crown sank 1.3 m. The slip surface was traced along its outcrops and in several observation boreholes. An attempt to stabilize the subsoil by means of horizontal boreholes (b) and a berm (f) did not succeed.

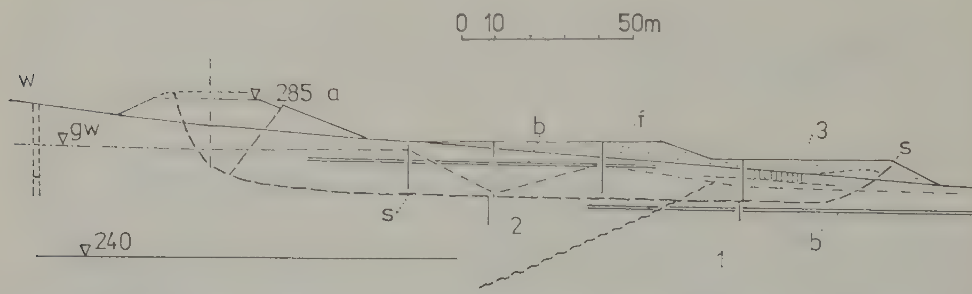


Fig. 11–16. Cross-section of the failed embankment at Buchlovice, Moravia; 1 — shales and sandstones (Eocene), 2 — mottled clays with sand layers (Pliocene), 3 — displaced loess loam, a — final elevation of the embankment, b — drainage boreholes, f — stabilizing berm, s — slip surface found in exploratory boreholes, w — house well.

A good drainage output was obtained from the holes bored into the Eocene layers, but the drainage from the Pliocene deposits was very poor. In spite of the drilling of more than twenty boreholes, movements of the water table in the observation holes were only sporadic, and striking differences were observed between levels in neighbouring holes. During the several stages of the treatment, the sunken crown of the embankment was built up three times, but each time it sank again, albeit with decreasing intensity on each successive occasion. The final settlement was only 0.6 m. The urgency of putting the road into service led to the lowering of the road by 2.5 m,

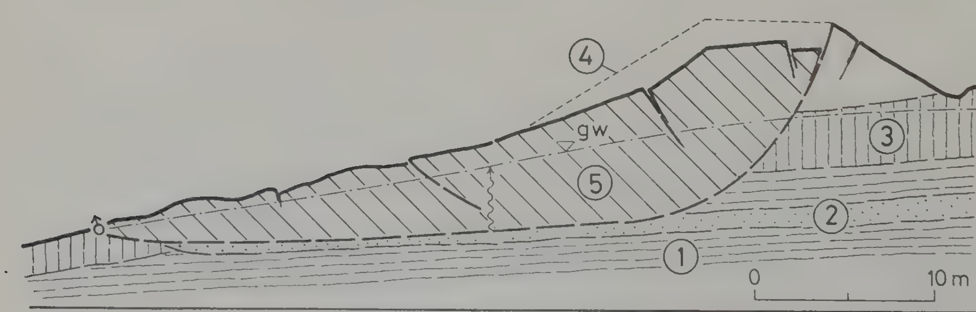


Fig. 11–17. Spreading failure of the subsoil of an embankment caused by the pressure of groundwater in a layer of Neogene sand (2); 1 — Neogene clay, 3 — loess loam, 4 — original embankment, 5 — collapsed portion of the embankment.

and since this was carried out the movements have not reappeared. The only benefit derived from the three unsuccessful attempts to reach the full height of the embankment was that the residual shear resistance was estimated to a good approximation (section 6.2.5).

Sometimes geological conditions such as this give rise to a spreading failure of the subsoil (Fig. 11–17). The downward movement of the mass spreads the uplift effect for some distance away from the embankment, so that the disturbance of the slope advances down the hill.

11.4.3 *Weakness of the subsoil caused by the uplift effect of ground-water supplied from underlying permeable strata*

This phenomenon has already been underlined in section 11.1, and is probably more widespread than is generally realized. It is of common occurrence in the Carpathian Flysch area. In a dry season, deep exploratory boreholes do not reach the ground-water level at all, but after heavy rains the water overflows at the mouths of the boreholes. Fig. 11–18 shows an example of this kind of failure involving the collapse of part of an embankment during the construction of a road near Vizovice in

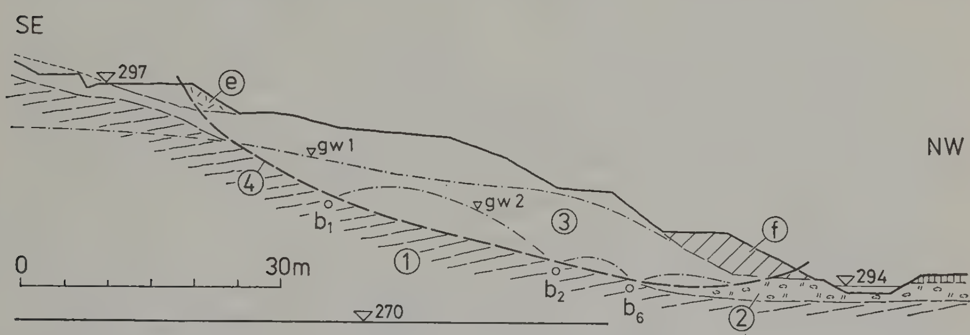


Fig. 11–18. Failure of the road embankment near Vizovice in Moravia; 1 — shales with sandstones (Carpathian Flysch), 2 — terrace deposits with boulders, 3 — old slide of debris, revived by the embankment loading, gw 1, gw 2 — original and drained ground-water tables, 4 — slip surface, e — embankment made of ash, b — horizontal boreholes, f — supporting fill.

Moravia. A thorough investigation was carried out, because a costly stabilization was at stake. The ground-water table, as plotted in Fig. 11–18, indicated clearly that the slope was supplied with water from the permeable sandstone beds. The intention was to support the road embankment with an anchored pile wall if the other measures (viz. drainage boreholes and a supporting fill) were to fail. Fortunately, this was not necessary, partly because ash had been used to fill the sunken part of the embankment.

11.4.4 Embankments on steep slopes covered with unstable debris

First of all, the hydrogeological conditions of the site must be studied because outflows of ground-water contribute to poor stability of the slope. Small corrections made to the design (Fig. 11–19) may be efficient in improving a situation, but often the conditions are so bad that artificial structures are necessary. Reducing the mass is an approach which was adopted on a small scale in the case shown in Fig. 11–18, and which has been validated on a large scale in the construction of the highway referred to in Fig. 11–20.

Overbridging is another, more widely adopted solution on steep slopes, exemplified by the construction of a motorway between Innsbruck and the Brennerpass in Austria.

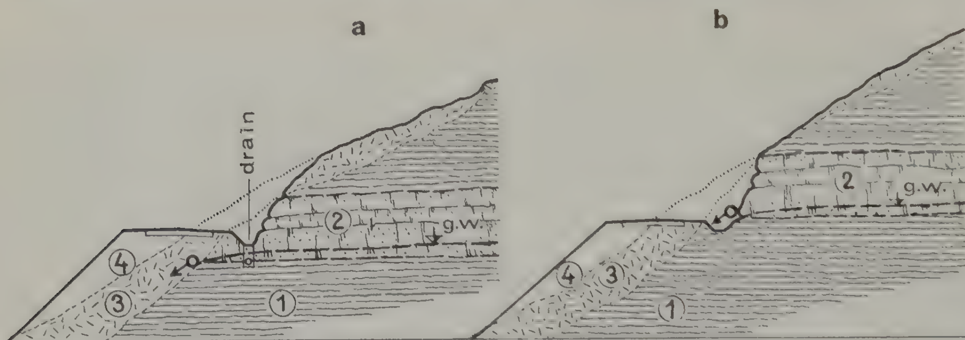


Fig. 11–19. (a) Ground-water flowing out from a sandstone weakens the base of the embankment and must be drained. By lowering the elevation of the road (b), water can be drained by a ditch cut in the impermeable marl; 1 — marl, 2 — sandstone, 3 — slope debris, 4 — fill.

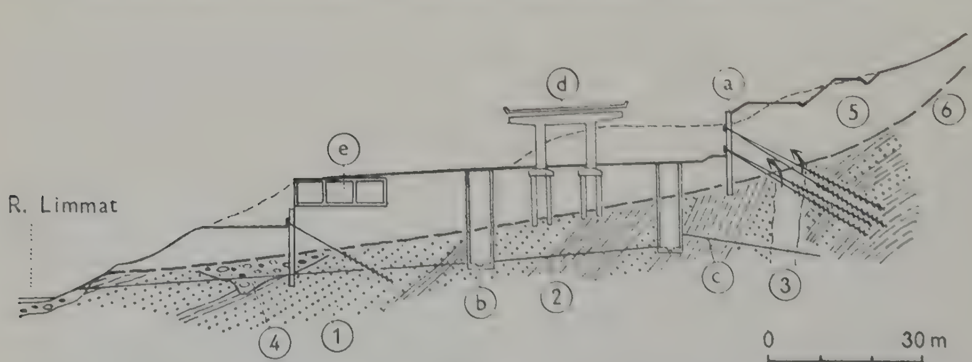


Fig. 11–20. Complex stabilization of a slide-affected area on the route of a motorway near Neuenhof Baden in Switzerland; 1 — sandstone, 2 — marls (Tertiary molasse), 3 — fault zone, 4 — sand and gravel of the abandoned Limmat channel, 5 — clayey sandy material of ancient landslide, 6 — slide surface; a — anchored pile wall, b — pumping wells, c — drainage boreholes, d — bridge construction, e — branch road built on a box structure so as to decrease the load on the slope (after Schindler and Gautschi, 1968).

A section of the roadway on the steep slope of the valley of the river Sill, which consists of phyllite and gneiss, tectonically disturbed and often even mylonitised, was supported by circular hollow piers. In several sections where the creep of rock debris presented a threat to the stability of the ground, the piers were founded deep in the rock and a space was left around them. This allowed a movement of debris of 1.7 m without the piers being affected.

11.5 Tunnels

Tunnels are often planned as an alternative means of dealing with dangerous slopes or situations in which the pressure on retaining walls of deep cuttings would be too great. In the example shown in Fig. 11–21, the route of a railway line near

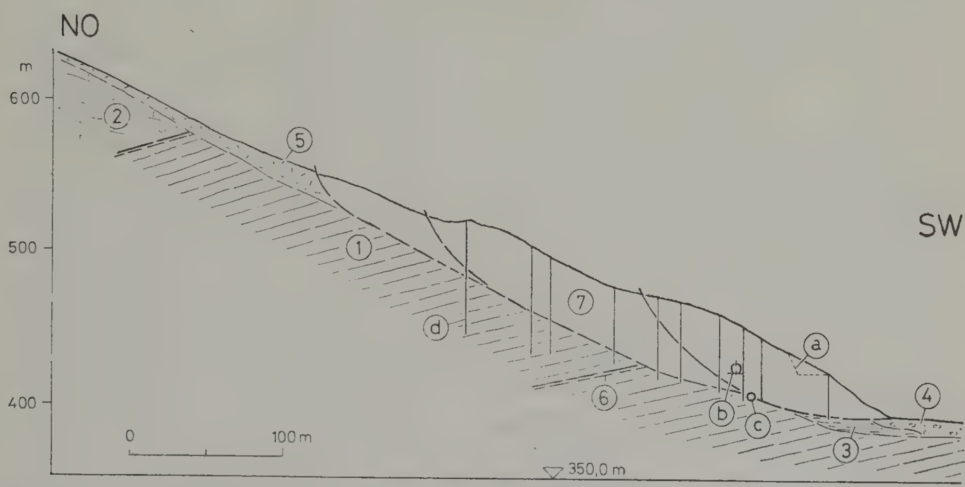


Fig. 11–21. Section through a mountain slope near Banská Bystrica in Slovakia, in which a railway tunnel was endangered by a landslide (Kuchár and Mencl 1977); 1 — Permian arkose, 2 — quartzites, conglomerates and limestones, Triassic, 3 — Neogene clay, 4 — alluvial gravel, 5 — debris, 6 — fault, 7 — fossil slope movement, a — abandoned hill-side cutting, b — railway tunnel, c — drainage gallery, d — exploratory boreholes.

Ulanka in Slovakia was to be taken through a hill-side cutting, containing Permian arkoses (Verrucano). These rocks, together with the underlying gneiss, form the base of a nappe and are therefore strongly tectonized. During the excavation an extensive but very slow slope movement began to occur. Three drainage galleries were sufficient to stop the movement, however any further excavation was deemed to be dangerous. The route had to be relocated in a tunnel, and a decision had to be taken as to how deep the tunnel should be in order to avoid the deep slope movement. Unfortunately, the less expensive solution was the more attractive and the first 100 m of the tunnel was taken through the sliding part of the ridge. The driving of the tunnel was difficult and caving-in occurred at the chainage where the shear zone was crossed. In the sixties the tunnel had to be repaired at this particular chainage. The investigation which was carried out very much with the drainage of the rock massif in mind, revealed the presence of Neogene deposits on the bottom of the valley as well as Quaternary gravel. This indicated that the slope was affected by successive phases of movement.

Chapter 12

LANDSLIDES AND URBAN PLANNING

A general principle that must be observed in planning the development of new towns or the redevelopment of existing urban areas is that the sites to be built up should be stable and not endangered by slope movements of any type. Anyone who had an opportunity to see the damage that sliding may cause to buildings and subsurface service lines, will agree that the main task of engineering-geological investigation is to recognize areas that have been disturbed by slope movements in the past and areas that are susceptible to sliding.

The founders of medieval towns avoided, where possible, building on hazardous slopes or valley floors threatened by floods. The older parts of towns are therefore usually situated on elevated river terraces, which provided a foundation of satisfactory stability and bearing capacity. The progressive growth of towns, particularly their abrupt and often chaotic development towards the end of the nineteenth century led to building on less desirable sites such as flood plains, or hillsides of low stability.

The effect of slope movements on the growth of a town is well demonstrated in the development of the city of Bath in England (Kellaway and Taylor 1968). In the eighteenth century, the medieval town built on a relatively stable ground of the valley floor started to expand up the valley sides, which are formed of Jurassic clays susceptible to sliding. Towards the end of the eighteenth century and during the nineteenth century there were serious and recurrent slope failures, which had to be stabilized by means of drainage galleries. In 1955 a Development Plan for the City of Bath

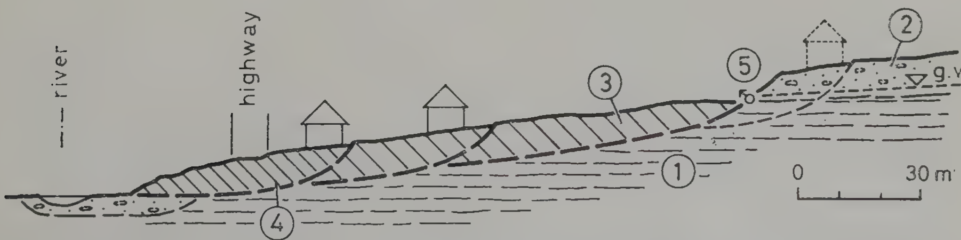


Fig. 12–1. Buildings constructed on a sliding slope in Turnov are threatened by recurrent movement; the house standing on a recent slide (4) had to be demolished; 1 — Turonian marls, 2 — sand and gravel (Pleistocene river terrace), 3 — ancient landslide, 4 — recent landslide, 5 — springs.

was drawn up, according to which the city's industry was to be concentrated on the valley floor and terrace sediments. The unstable slopes were redeveloped after deep drainage systems and piled foundations were installed. Some rather extensive areas should remain undeveloped and partly laid out as public gardens.

In Bohemia, several towns and villages in the Cretaceous region were also built and extended onto slopes threatened by sliding. It was probably because the irregularly undulating and wetted land was not suitable for farming that it was used as building sites which were cheap and easy to obtain. Numerous dwelling houses and farmsteads constructed on this land are repeatedly seriously damaged whenever there is high precipitation. This occurs, for example, in the town of Turnov (Záruba 1946), some quarters of which have suffered great damage (Fig. 12–1). A few villages in this area were so badly damaged (Holé vrchy – Hanuš et al. 1977) or extensively destroyed (Dneboh - Záruba et al. 1966) that they had to be demolished.

At present, as the population of towns and cities continues to increase at a great rate together with industrialization of the countryside, suitable building sites are generally becoming extremely scarce, especially where agricultural and forest land is protected by law against urbanization. As a result, planners are forced to consider less suitable sites, even those that have a potential for slope failure provided that they can be rendered safe by adequate means. Such hazardous slopes require very thorough engineering-geological investigation, especially in the reconnaissance stage.

12.1 Engineering-geological investigation

It must be emphasized that it is not sufficient only to study the stability of the immediate site; the investigation must cover a wider area, since structures may be endangered by slope movements initiated at some distance from the building site.

The objective of the reconnaissance investigation is to recognize the geological structure and the morphological forms of the landscape and their history. The morphology is predetermined by local geological conditions, i. e. by the character of rocks, their resistance to weathering and erosion, and the attitude of the surfaces of discontinuity – the strike and dip of bedding planes, joints and fault zones. The other important factors are volcanism, earthquakes and tectonics, which may have caused uplifts or subsidences, or other deformations.

The basic regional pattern was exposed to various exogenous processes, the principal agents in this respect being the water, climate and mass movements of diverse types. The physical processes have modelled the earth's surface, the final forms of which allow us to study the successive stages of its development. The study of surface forms, the inclination of slopes, the irregular undulation of the land surface etc., is the first step in the investigation of a landslide-prone area. However, it gives meaningful results only if the landscape has not been excessively reshaped by human interference.

The best approach in obtaining preliminary information about the occurrence and frequency of slope movements is to study geological maps, which are available in most countries today, and the geological literature pertaining to the area in question. For some heavily landslipped regions, landslide inventory maps and landslide inventories have been prepared. Air photographs of an adequate scale are useful for identifying slopes that have been disturbed by sliding, and help in delimiting the area that needs to be investigated.

This preparatory work is followed by field work, which involves determination of the type and size of the movements, estimation of their depth, investigation of the hydrogeological conditions, enquiry into the feasibility of drainage, etc.; a more detailed program of investigation can then be drawn up. The details of this research have been discussed in Chapter 6.

The effect that the intended buildings and other structures might have on natural conditions and slope stability has to be considered in the preliminary investigation, and represented on specified slope-stability maps. The planner thus obtains information about the type of slope movement involved, including its depth, age and activity.

The activity of landslides is largely governed by precipitation and snow melting, and thus changes on a seasonal basis, although there are also longer (e. g. 11-year) cycles of activity. Since the investigation cannot be usually conducted over such a long period, it is necessary to state whether it was made in a dry or a wet season. In a number of cases the results of geological research carried out in dry periods have led to erroneous and over-optimistic interpretations.

In the case of seismically active regions it should be borne in mind that earthquake shocks are one of the chief factors responsible for triggering slope movement and that they appreciably increase the potentiality for rockslides, even in places which would otherwise be stable. An investigation of slopes designated as building land must therefore include a study of the seismicity of the area.

12.2 Importance of the type of landslide in urban planning

Different types of slope movement threaten urban development schemes in different ways.

Areas affected by sheet slides of slope debris and residual material of little depth can in many cases be used for building, provided that the structures are founded at a sufficient depth on firm ground (e. g. by using piles), and that the slide area is well drained. Urban development may thus contribute to the stabilization of a hillside.

An example of how a sheet slide may be hazardous to a building as a result of the slope stability being disturbed by excessive interference, is given by the construction of a sanatorium in eastern Moravia. The structure was founded on firm sandstones and shales; the cutting was protected by a low retaining wall and the slope above it was excavated at a gradient of 1 : 1.5 (Fig. 12−2). During construction, the building was damaged by a sheet slide along its entire length. It had simply been overlooked that the slope was covered with slope debris susceptible to sliding. In the spring after the reinforced-concrete skeleton and facework had been erected, the debris started to move and the material filled the space between the retaining wall and the building,

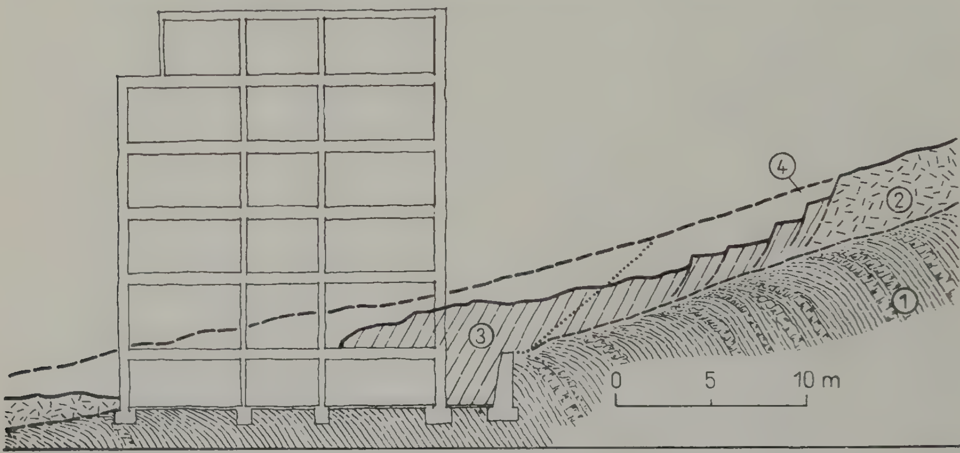


Fig. 12–2. Building damaged by sliding of slope debris, Luhačovice (Moravia); 1 — argillaceous shales and sandstones (Palaeogene); 2 — clayey-sandy slope debris, 3 — slid material, 4 — original slope surface.

breaking through the facework of the ground floor. Fortunately, the concrete structure was not damaged, as the slaked material flowed onto the ground floor around the columns. Repairs were easily effected by removing the displaced material, stabilizing the slope by means of several drainage trenches, and reducing its inclination.

In contrast, valleys in mountain regions which are either filled with earthflows or threatened by debris flows whenever there is torrential rain, are unsuitable as building land. This is a particularly valid point in cases where there is still a thick layer of slope debris above the head scarp in the upper part of the valley; such a layer of debris can easily be set in motion causing upward extension of the head area.

The debris flows triggered by heavy rains are a potential hazard particularly where large parts of mountain slopes have been deforested. This should be borne in mind if a development of a dejection fan is intended, being otherwise a suitable site. Nasmith and Mercer (1979) refer to a serious damage which the town of Port Alice on Vancouver island in Canada suffered from two successive debris flows. The town was built in the sixties on a dejection fan at the foot of a steep slope some 900 m high (Fig. 12–3). The slope formed of Triassic argillites and limestones with a thin mantle of overburden is cut by three gullies 10–20 m deep. The first debris flow occurred on December 15, 1973 after a torrential rain (120 mm in 24 hours) and a sudden thawing of snow in the upper part of the slope. The water mixed with soil,

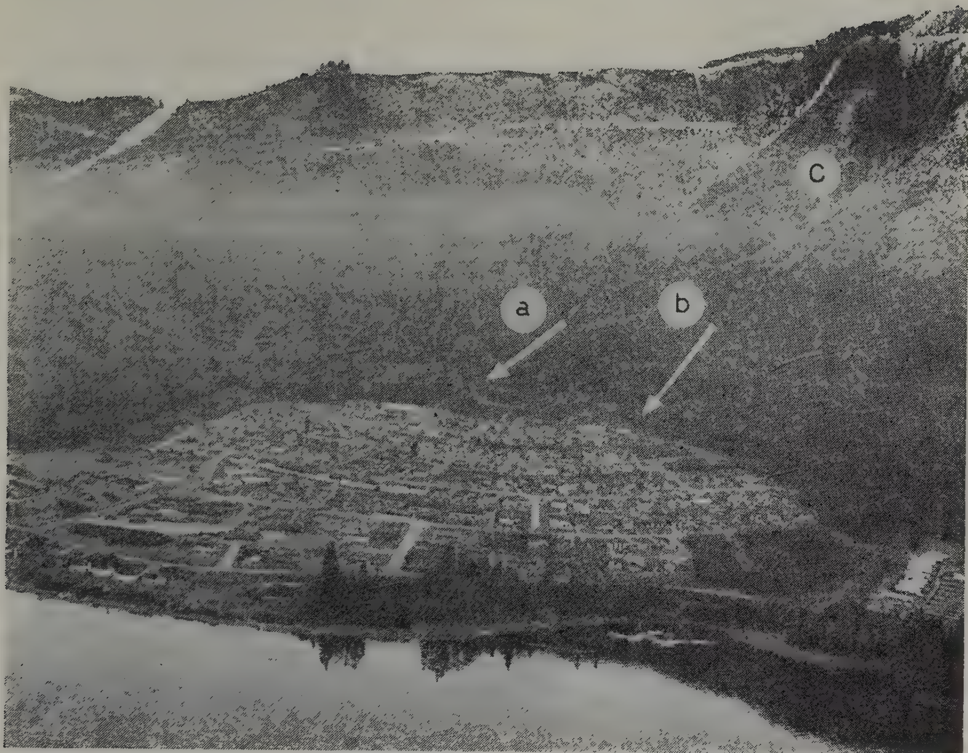


Fig. 12–3. Oblique aerial view of Port Alice Townside showing the fan on which the town is built and the steep slope behind the town; a, b — gullies in which the debris flows of 1973 and 1975 developed, c — logged area (courtesy of Nasmith and Mercer).

rocks and forest debris concentrated to gully *a* (Fig. 12–4) and moving down it swept material from the gully. The debris flow came to rest in the upper part of the fan and damaged several houses. The amount of debris deposited in the town was estimated at 22,000 m^3. The second debris flow of November 12, 1975 was again preceded by a downpour reaching 170 mm in 24 hours. The water laden with debris moved as a true flow down the gully *b*. The debris flow stopped again in the upper part of the town and caused considerable damage. It seems that heavy logging on the upper slopes contributed to these disastrous events. To provide permanent protection for the town a system of dykes was built of coarse material of the fan. The position of the dykes (c in Fig. 12–4) was determined from the results of model tests.

Enormous debris flows are known to occur in mountain areas of Central Asia where they threaten towns and villages in deep valleys. During the last hundred years the town of Alma Ata in Kazakhstan, e. g., was severely damaged by debris flows. In order to protect the town an earth dam 140 m high was built across the valley. In 1973 the dam not yet completed retained a debris flow which brought four million m^3 bouldery debris into the reservoir, thus saving the town from further disaster (Niyasov 1975).

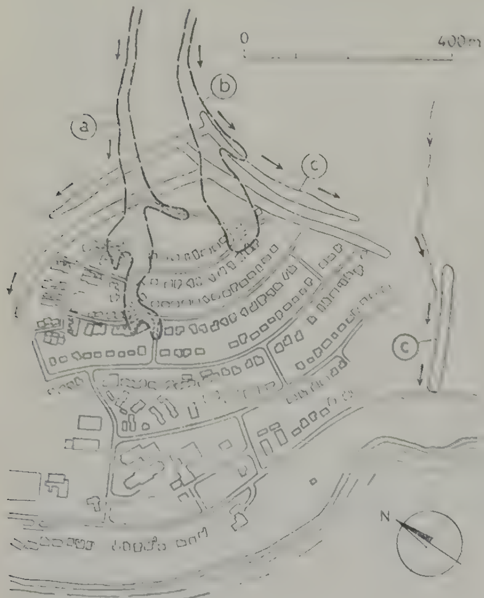


Fig. 12–4. Plan showing the debris flows that threatened the town of Port Alice in 1973 and 1975, and the system of protective dykes; a — debris flow of 1973, b — debris flow of 1975, c — protective dykes (after Nasmith and Mercer, 1979).

Slumping along rotational slip surfaces usually reach deep under the ground surface and any corrective operations would be so expensive that they could be economically justified only in exceptional cases. As an example we may cite the location of a new township near Brno (Moravia) on a slope of Neogene clay, which was disturbed by several rotational slides. It had to be stabilized by a system of horizontal drainage borings (Mencl et al. 1977). A community situated unsuitably at the foot of a slope disturbed by repeated slumping may be typified by the village of Podháj (Bohemia). The slope is today at rest but the village may be damaged by slumping whenever the

Fig. 12–5. Podháj village threatened by reactivated sliding movements following heavy rainfall; 1 — Turonian sandy marls, 2 — glauconitic sandstone, 3 — Senonian marls, 4 — Senonian thick-bedded sandstone, 5 — kneaded marls along slip surface, 6 — slipped sandy debris.

weight of the debris in the head area is increased after heavy rainfall or spring thawing (Fig. 12–5).

Sites below rock walls susceptible to rockslides and rockfalls are also unsuitable for development, chiefly in seismically active regions. A number of examples are given in Chapter 5.

Assessing the suitability of areas disturbed by block slides as potential building sites is somewhat more complicated. Under the present climatic conditions of Central Europe, most block slides are stable, the movements having been active during the thawing of the perenially frozen ground in Pleistocene times, some of them probably having been triggered by earthquakes. Only in some mountain regions have recent slow (15–40 mm per year) movements of individual blocks been measured locally (Pašek and Košťák 1979).

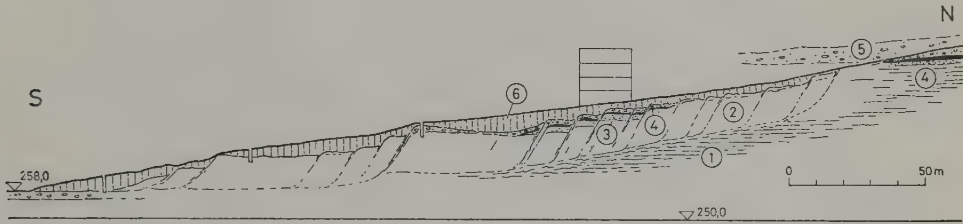


Fig. 12–6. Hospital founded on a Pleistocene block slide which has not shown any deformations since it was built 40 years ago; 1 — Turonian sandy marls, 2 — inclined blocks, 3 — friable sandstone, 4 — green glauconitic sandstone, 5 — terrace sand and gravel, 6 — loess loam.

In 1942, a hospital several storeys high in Turnov was founded on a block slide consisting of a system of tilted blocks of Cretaceous marl (Fig. 12–6) and so far the slide has not shown any sign of movement (Záruba 1976). On the other hand, block slides had catastrophic results in the City of Algiers in 1943 (Fig. 5–44) and according to historical records they have occurred repeatedly in the past (1829, 1845, 1942 – Agard 1948; Drouhin et al. 1948).

Areas suceptible to rapid earthflows in fine-grained sensitive silts and clays are extremely dangerous. The characteristic feature of these quick-clays is an abrupt and considerable loss of shear strength on remolding, which results in liquefaction. Most of them are of marine origin and form terraces on elevated coasts in Scandinavia, Canada and Alaska. They are dangerous mainly because movement may occur even on a level area as a result of vibration or undercutting. Towns situated in areas in which sensitive clays are present are in serious danger particularly if the area becomes seismically active. During the great earthquake in Alaska in 1964 the town of Anchorage (Fig. 5–69) suffered great damage directly from slides of sensitive clays (Hansen 1965; Dobrovolný and Schmoll 1968).

Ancient landslides buried by younger sediments (e. g. loess or slope detritus) are difficult to recognize. Even fossil slides of Pleistocene age may be reactivated by careless interference, such as excessive loading of the head area, or excavation of the toe of the slide in the accumulation area.

Since landslides are phenomena that change with time (e. g. slumps expand retrogressively upslope and develop into earthflows in the direction of movement), investigations should be started well ahead of the construction work, and as far as possible the effect of corrective operations should be observed after construction work has been completed.

12.3 Town planning and geological conditions

Plans for building on a hillside in a landslide-prone area must respect the local geological and morphological conditions, which may effectively restrain the growth of a town. The location of houses and the laying-out of transportation routes have to be adapted to the environment. Many plans are prepared with a view to restrict the necessary earthwork to a minimum and to attain the easiest access to the upper part of the slope regardless of the geological setting. When such plans are realized, the construction costs and subsequent maintenance costs tend to escalate.

When permission to build according to an inadequate planning scheme is given, a complicated legal situation may arise, since the authorities concerned have a legal responsibility towards the builder. Moreover, the installation of service mains and other subsurface services may upset the state of equilibrium of the entire slope. To avoid these complications a professional engineering-geological report is required in some countries before building permission is granted.

In San Mateo County in California, for example, a landslide susceptibility map and a landslide inventory have been compiled, indicating those areas that require to be geologically investigated before being developed. On the basis of this map a decree has been issued reducing the density of buildings to one dwelling unit per 16 hectares in areas having a high potential for slope failure. An engineering-geological study of the site is mandatory, even for the construction of one dwelling unit (Nilsen, Brabb 1977).

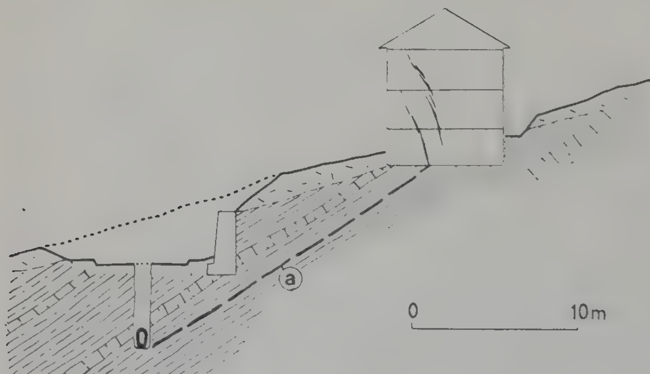


Fig. 12–7. The lay-out of streets should take into account the dip and strike of the beds. Excavation for sewerage brought about the movement of shales along bedding surface *a*.

If it happens that a site liable to slope failure is essential to the development of a town and cannot be excluded from the planning scheme, then stabilization of the slope should be carried out well in advance of construction work.

Owing to the diversified relief of the area, several quarters of the city of Prague were built on steep slopes, and many slope failures were triggered during the period

Fig. 12-8. Open fissures in Cenomanian sandstones at the margin of the Cretaceous plateau in the suburb of Prague (see Fig. 12-9); photograph by Rybář.

of construction. The valley slopes are formed predominantly of Ordovician shales, the stability of which depends upon the strike and dip of the beds relative to the inclination of the slope. Where streets were laid out parallel to the strike of beds dipping downslope, excavations of road cuttings usually gave rise to sliding movements along the bedding surfaces. Excavations for subsurface lines, particularly those for deep-lying sewers, also resulted in rock sliding (Fig. 12–7).

Increased attention must be given to the margin of elevated Cretaceous plateaus where sandstone layers of considerable thickness overlie beds of soft marl. The sandstones are cut by vertical open fissures separating the blocks (Fig. 12–8); these incline gradually away from the edge of the plateau and tend to slip downward (Fig. 12–9). An engineering-geological investigation for a new district to the north

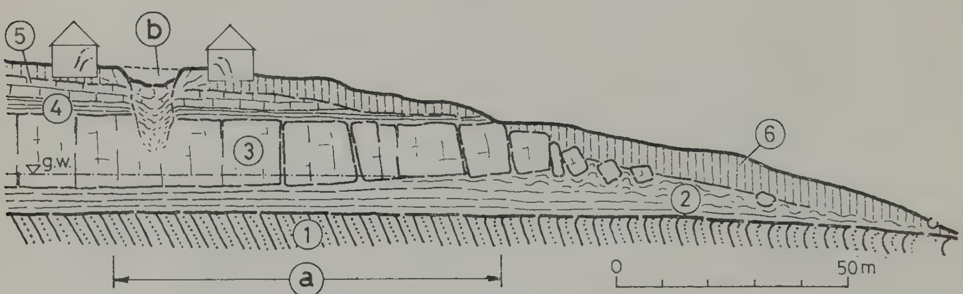


Fig. 12–9. The edges of Cretaceous plateaus are not suitable for building purposes. The sandstones are disrupted by open fissures, old slope movements and locally by the underground working of sand; a — undermined area, b — subsided street, 1 — Ordovician shales, 2 — Cenomanian claystones, 3 — sandstones, 4 — Turonian marls, 5 — firm sandy marls, 6 — slope debris and man-made ground.

of Prague revealed that the stability of the blocks had been disturbed in places by exploitation of the sandstone, and it was therefore recommended that the margin of the Cretaceous plateau should remain undeveloped and preferably reserved as a "green belt" of parks, gardens and sport fields (Záruba 1968, 1976).

Whilst there may be no constraints on the architect's plans for developing flat, level areas such as river terraces with good foundation ground, the planner has to pay heed to the results of engineering-geological investigation when the building site involves an unstable slope. Otherwise, he may meet with serious difficulties resulting in increased cost, prolongation of the period of construction, and sometimes considerable damage to property. Thus the development of landslide-prone areas requires close co-operation between the planner and engineering-geologist right from the beginning, so that the geologist can become acquainted with the intentions of the planner, and the planner can be well informed about the geological setting of the building site.

In selecting areas for city development, serious attention must be given to abandoned quarries and pits left after the exploitation of mineral deposits, and then more or less filled-in with refuse and waste. Unfortunately, old quarries are rarely reclaimed after the mineral resources have been exhausted unless they are easily accessible and suitable for the deposition of waste material. The walls of quarries and loam pits are very often left so steep that they cannot remain permanently stable. This means that the zone adjacent to an abandoned quarry is always in danger from potential landslides, both at the foot and at the upper edge of the wall.

In some cases abandoned quarries have been used as building sites, usually because the increasing lack of suitable land forced the planners into this course of action. In the northwestern district of Prague a terrace block of flats was built on a slope that had been a wall of a sandstone quarry (Fig. 12–10). Geological investigation showed that the marginal sandstone blocks had sunk several metres into the underlying

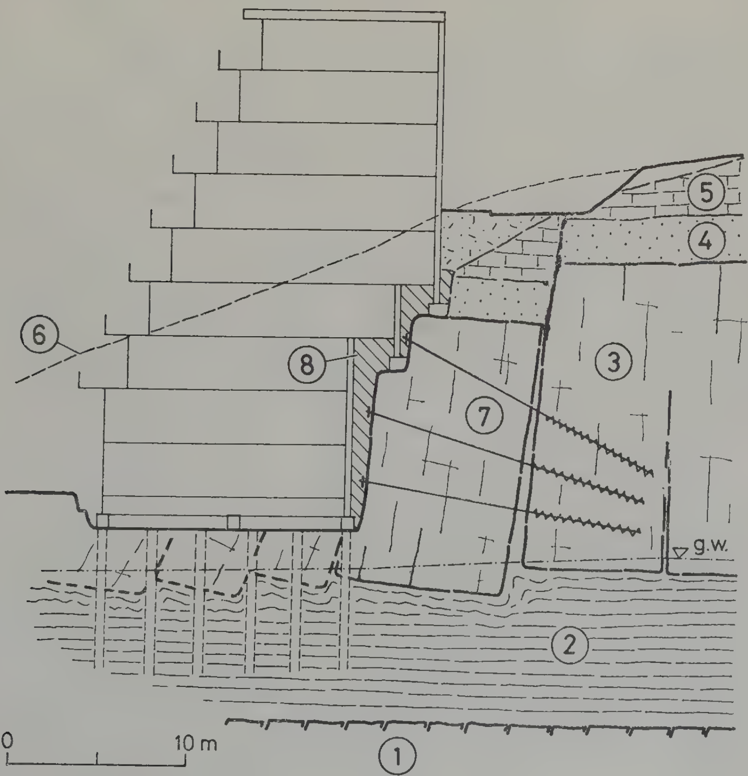


Fig. 12–10. Construction of terraced blocks of flats in an abandoned quarry; 1 — Ordovician shales, 2 — Cenomanian clays, 3 — Cenomanian sandstones, 4 — glauconitic sandstone, 5 — Turonian marls, 6 — original slope surface, 7 — steel anchors, 8 — concrete back-fill.

Cretaceous clays, suggesting the existence of an ancient block-type landslide (see Chapter 5). The amount of movement was indicated by the course of green glauconitic sandstone layers. Since the thickness of the sandstone below the intended foundations was relatively small and the rock was largely disintegrated to sand, which was water-bearing at the base, the frontal part of the houses was founded on reinforced-concrete cast-in-place piles; the piles were driven as deep as the solid claystones. The sunken sandstone blocks were secured by steel anchors about 16 m long, prestressed to 200 kN (20 tons).

A collapse of the wall of an ancient loam pit in the northwestern part of Prague is shown in Fig. 12–11, and Fig. 12–12 portrays a rockslide that badly damaged buildings built too near a steep quarry wall.

As part of any development scheme, it is therefore necessary to provide for the reclamation of ancient quarries. Some may be filled-in, while the walls of others may be modified so that slopes of permanent stability are formed; a sufficiently broad belt along the edge of the quarry should remain undeveloped. Those quarries which exhibit interesting rock formations may be reserved as parks or places of geological interest.

Fig. 12–11. Collapse of a steep slope of an abandoned loam pit in a suburb of Prague. Streets and buildings in the neighbourhood were damaged (photograph by Záruba).

High waste heaps with unstable slopes also require corrective treatment. Large-scale slides of waste heaps occur, for example, in the Most brown-coal basin in Bohemia, where the heaps are usually piled up on Tertiary clays of small bearing capacity. There are a menace to transportation routes, streams, and housing developments.

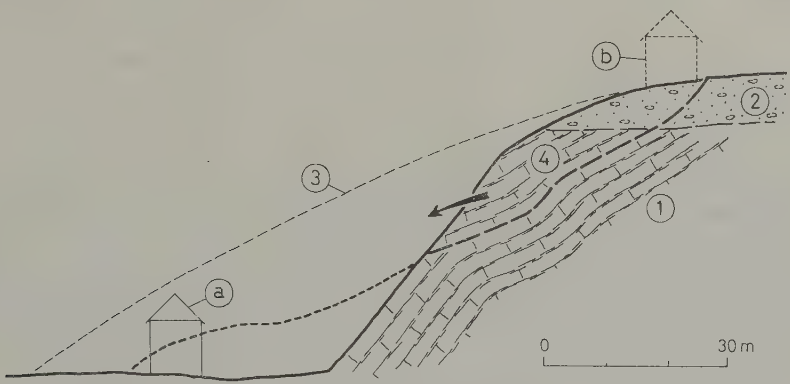


Fig. 12–12. The buildings (a) below and (b) above the steep wall of a disused quarry were damaged by a rockslide; 1 — slabby limestones with shale interbeds (Devonian), 2 — sand and gravel (river terrace), 3 — original slope surface, 4 — displaced limestone block.

Statistical data on the increasing populations of large cities in recent decades indicate that at the turn of the century about one half of the world's population will live in cities of more than 100,000 inhabitants. If this is so, it will be necessary to further extend urban development over huge areas. Legget (1973) reports that in the U.S.A. alone the growth of the cities will require 360,000 km^2 of land by the end of the twentieth century. Structures will have to be built on difficult sites, including slopes susceptible to sliding, and therefore the problem of finding reasonable and adequate means of developing landslide-prone areas is a very urgent one.

Chapter 13

MASS MOVEMENTS AND THE EXPLOITATION OF MINERAL DEPOSITS

The exploitation of natural resources under the ground has often caused slope movements at the ground surface. An example of a disastrous collapse of this kind was referred to earlier (Fig. 5–65). Even the lesser of such movements may threaten not only the activity of the mine itself, but also nearby structures (buildings, roads, sewers) and even streams.

13.1 Underground mining

Underground mining may provoke subsidence of the ground surface within an area limited by the angle of draw (about 20° from the vertical). Earth movements, however, may occur beyond this area, usually without reaching to the depth of the exploited seam. They are caused by two factors:

(a) by an increasing difference in height between the levels of the ground surface as a result of subsidence,

(b) and/or by the loosening of soils or rocks in areas of tension surrounding the subsided area.

In both cases extensive slope movements may develop where weak soils or rocks are present and where hydrogeological conditions are conducive to slope failure. Fig. 13–1 shows a cross-section (Malgot and Mahr 1979) of a site not far from that shown in Fig. 4–18. The Miocene coal seam and the overlying claystones extend beneath the loosened margin of the andesite flow. As suggested in section 5.2.3, the loosening of a marginal zone such as this has the character of a creep movement. In this particular case, the movement was revived by the mining operations, and several buildings were damaged although they were situated far from the mined seam. The depth of the sliding zone as compared with that of the roof of the seam was dangerous for the mining.

Sometimes detailed records of the extent of former underground mine workings are lacking, often because only limited areas were worked by the owners who quickly abandoned the mine on account of flooding or some other natural impediment. An example of this is shown in Fig. 13–2, from the border of an open pit coal mine in northern Bohemia. The Miocene clay and sand layers containing the coal seam lie on the strongly kaolinised paragneiss on the border of the basin. In the course of the open pit excavations displacements of the coal seam became apparent and tensile stresses of the order of 0.05 to 0.1 MPa were measured beyond the top of the slope. It is noteworthy that the general angle of inclination of the slope was 10°. After mining operations had progressed a little further, a head scarp developed. At the

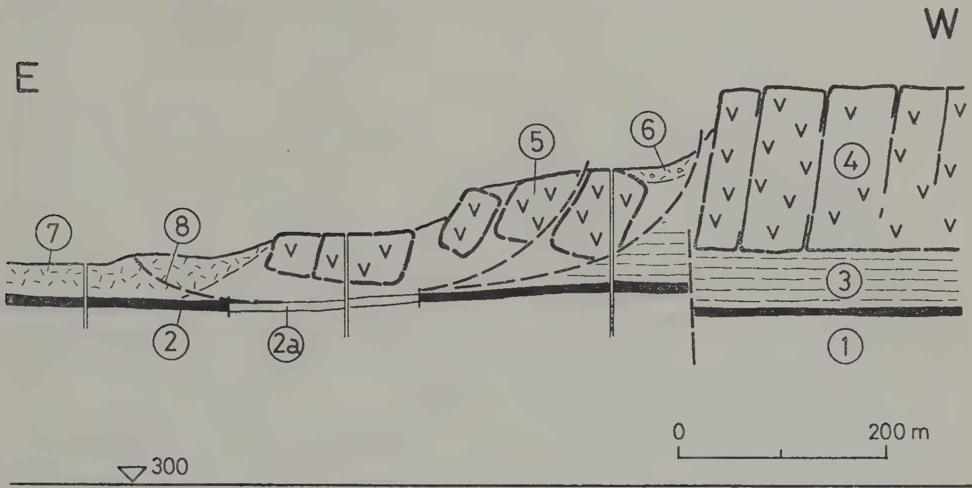


Fig. 13–1. Cross-section of the marginal part of the Neogene Nitra Basin in Slovakia; 1 — tuffaceous deposits and coaly clay, 2 — main coal seam, 2a — mined seam, 3 — overlying clay, 4 — andesite, 5 — slipped andesite blocks, 6 — slope debris, 7 — Quaternary material, 8 — assumed position of slip zones (Malgot and Mahr 1979).

same time an ancient adit was found near the base of the seam, discharging water at 0.9 m^3 per minute for several hours; even after three years the measured rate of flow was 0.3 to 0.5 m^3 per minute. It is very likely, therefore, that the mining had to be abandoned about 50 years ago because of the inflow of water. Although 30 long drainage boreholes and 18 deep pumping wells were installed in the slope and its

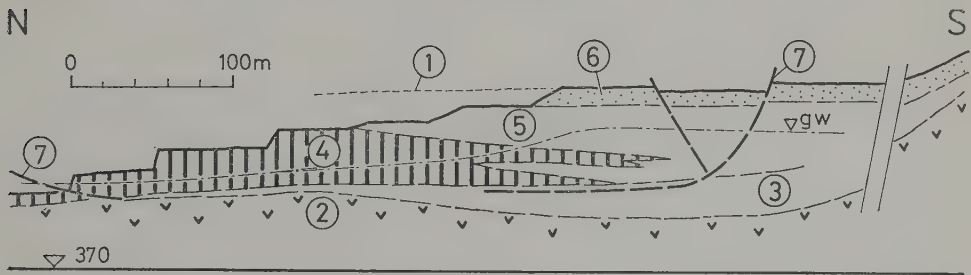


Fig. 13–2. Profile of the southern margin of the Sokolov coal basin, Bohemia; 1 — original ground surface, 2 — kaolinized paragneiss, 3 — tuffaceous clay and fine sand, 4 — coal seam, 5 — sand and clay, 6 — alluvial fan, 7 — slip surface.

surroundings, the discharge from these did not reach more than 0.2 m^3 per minute and an effective lowering of ground-water table was not achieved. In spite of extensive hydrogeological investigation, no water-bearing layer could be found. The original miners possibly came near to a large fault which crosses the underlying decomposed gneiss in the N–S direction and which manifests at the surface as a narrow valley in the hills south of the basin. Owing to the presence nearby of an important road and the outskirts of a town to the south of the mine, careful exploitation was necessary. Cuts were made in a direction normal to the slope face and as they progressed worked-out cuts were filled in.

13.2 Opencast coal mining

The problems of slope safety when excavation is carried out within the basin are different from those which occur on the margins of the basin. Also, the approach to the temporary working slopes is different from the approach to creation of permanent slopes. Before discussing the problems in detail, two observations concerning static conditions need to be made.

13.2.1 Peculiarities of the static equilibrium of coal layers

Workers experienced in the problems of slope safety can usually form a good judgement of a situation on the basis of a simple profile of the slope. However, these judgements may be erroneous if coal layers are involved, basically for two reasons:

(a) Brown coal has a low unit weight. A theoretical relationship belonging to the subject of soil mechanics (it is not entirely valid in practical use) maintains that the safety factor of a soil layer inclined at an angle β, and possessing an angle of shear resistance φ, is given by $F = (1 - r\gamma_w/\gamma) \tan\varphi/\tan\beta$, when the layer is subjected to uplift forces given by the piezometric height rh, h being the thickness of the layer. This relationship is derived from the equilibrium of forces acting on the layer (Fig. 13–3); γ is the unit weight of the soil and γ_w is the unit weight of water.

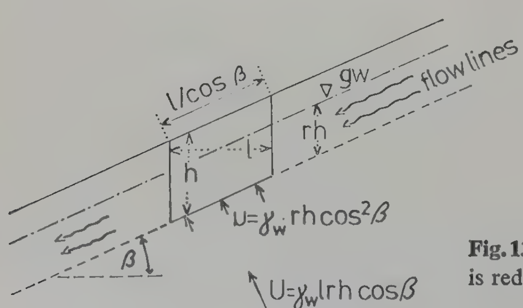


Fig. 13–3. The safety factor in shear of a coal seam is reduced by uplift to a much greater extent than that in shear of rock and soil.

For soils ($\gamma = 20$ kN m^{-3}, $\gamma_w = 10$ kN m^{-3}) the formula yields:

for $\quad r = 0 \qquad r = 0.5 \qquad r = 1$

$$F = 1.0 \tan\varphi/\tan\beta \qquad 0.75 \tan\varphi/\tan\beta \qquad 0.5 \tan\varphi/\tan\beta$$

For a material with $\gamma = 10$ kN m^{-3}

for $\quad r = 0 \qquad r = 0.5 \qquad r = 1$

$$F = 1.0 \tan\varphi/\tan\beta \qquad 0.5 \tan\varphi/\tan\beta \qquad 0$$

The unit weight of brown coal is greater than 10 kNm, but still the comparison of the two rows of safety factors is instructive.

(b) Tertiary coal seams have a low shear strength along clayey interbeds or at the interface with clayey bedrock. Experience with open pit mines in Czechoslovakia indicates that the angle of total shear strength (in the absence of ground-water and under unloading conditions) is 8°, despite the fact that for the underlying clay this angle appears to be 20°. Seyček (in Mencl and Seyček, 1975) measured a shear strength angle of 20° to 38° for the interbedded clay in a coal seam (shearing parallel to the bedding). Metchkarski et al. (1977) suggested that the small shear resistance of brown coal could be ascribed to its low Bingham threshold. The process of instantaneous creep begins at a shear stress of about 20 to 40% of the classical shear strength. Kazda (1976, unpublished report) explains the anomalies in terms of the presence of extremely thin organic colloidal interlayers in clay near the zone of contact with the coal. But there is also some possibility that gas pressure generated by the coal might be a factor involved in the reduction of the effective stress, since laboratory tests indicate greater values for the shear strength.

The findings of Rybář (1971) in mines in Bulgaria (Fig. 13–4) suggest that it was the low shear resistance that allowed the coal seam to be moved uphill, the angle of the line connecting the outcrops of the slip surface being only 4.5°.

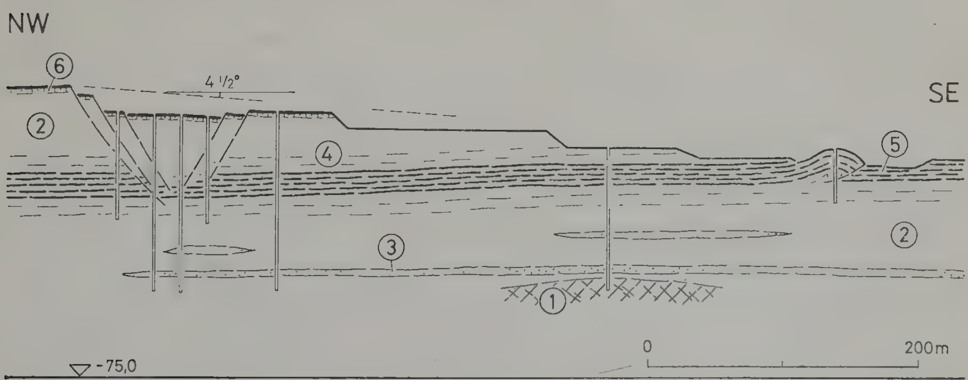


Fig. 13–4. Collapse of a slope in the Marica-Iztok mine in Bulgaria; 1 — crystalline rocks, 2 — clay, 3 — sand, 4 — coal clay, 5 — brown coal, 6 — Quaternary (Rybář 1971).

Back-calculation from the stability of the slope shown in Fig. 13–2 yields $\varphi' = 12°$ for the angle of effective shear strength of the coal clayey interlayers, when the corresponding angle for the clayey fine sands has been assumed to be 26°. This seems to support the tradition in Czechoslovakia of ascribing shear angle of 7° to the sloping clay seams, and an angle of 14° to the horizontal-bedded seams. The latter value seems to be too large when compared with the situation shown in Fig. 13–2 in which the creep movement continued at an angle of 12°.

The poor effect of drainage on the stability of brown coal seams is attributable to this low angle of shear resistance. Moreover, if the hypothesis advocating an effect of gas pressure is correct, it may be that the latter effect takes the place of uplift as the principal factor in the reduction of shear strength.

13.2.2 Temporary working slopes

In brown-coal opencast mines the temporary working slopes are influenced mostly by two factors. The first of these is the technique adopted for excavating banks of clay or other material, with due regard being given to the safety of the heavy excavators. The second factor is the required width of the cuts to accommodate the operational width of the excavator and space for through tracks. A very small overall angle is needed for these slopes, less than 10°. The angles of the slopes of individual cuts are determined by the working characteristics of the excavator. Digging-wheel excavators produce a rather steep slope, which is later reduced to a gentler gradient by bulldozers. There is therefore something of a risk involved, although the site engineers are usually armed with a great deal of local experience. The slope is surveyed in advance in order to identify possible planes of weakness. Pocket penetrometers are used to test the stiffness of the clay, and routine formulae are applied in checking the stability. If necessary, the height of a slope can be decreased by grading the upper part of the face.

13.2.3 Final working slopes inside the coal basin

After a seam has been worked, the slope will be covered with waste material but since this will take several years, the stability of the slope can be regarded as a long-term problem. Such a "final" working slope inside a coal basin is the slope that will remain unchanged after an obstacle, for example a river, limits further expansion of the pit. Therefore, in contrast to the foregoing, the steepest possible slope is aimed for.

An example of the static analysis with respect to drained slopes was given in section 7.9. Profile B in Fig. 13–5 illustrates the problem. If drainage cannot be effected, as is often the case because of the presence of fine silty sand, the slope angle must be reduced even though this will not guarantee slope stability. An example is shown in Fig. 13–6 in which overlying clay layers (6) displayed small movements at a slope angle of 10°, mostly as a result of the seepage of ground-water above the coal seam. A small weighting fill was necessary. The topography of the excavated pit allowed the

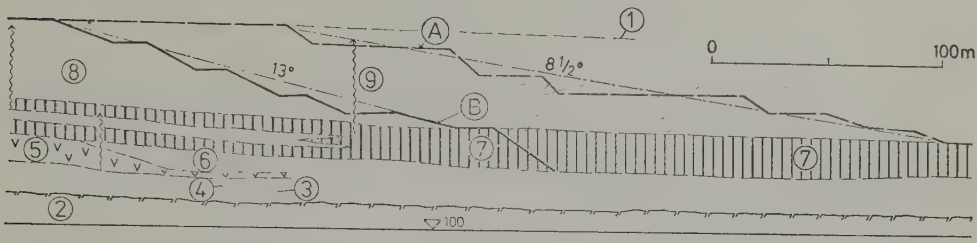


Fig. 13–5. A typical distinction between a provisional working slope (A) and quasi-permanent slope (B) in opencast mines; 1 — original ground surface, 2 — kaolinized gneiss, 3 — sand, 4 — kaolinitic clay, 5 — tuffaceous clay, 6 — underlying clay, 7 — coal seam with clay interseams, 8 — overlying silty claystone, 9 — uplift of water measured in exploratory boreholes.

boring of horizontal drains into the flank, and these were extended as far as possible along the developing slip surface. These boreholes yielded 12 1/min which was a good rate of discharge under the conditions of a brown coal area. Probably it reached a fissure which was created by the movement of the slope and which opened owing to the arching of the mass above (section 4.8).

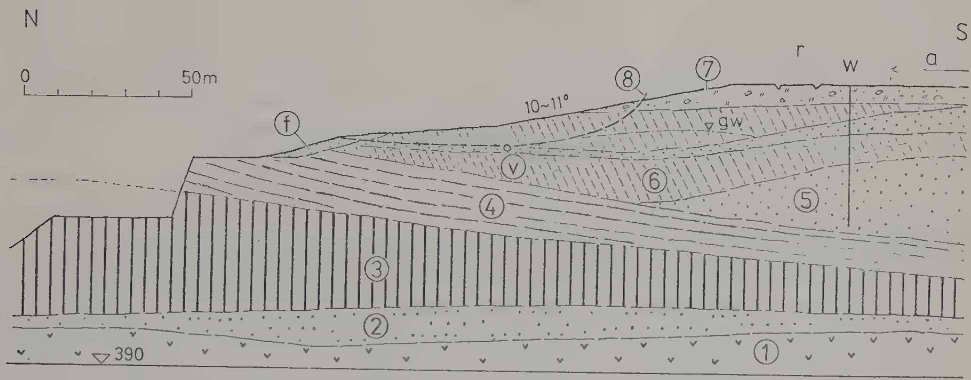


Fig. 13–6. This high slope of an opencast mine near Sokolov in North Bohemia suffered slow deformation even at an angle of 10—11°. The ground-water table could not be lowered satisfactorily either by pumping wells (w) or by horizontal borings (v); 1 — kaolinized gneiss, 2 — sand, 3 — coal seam, 4 — clayey coal, 5 — sand layer, 6 — clay to claystone, 7 — Quaternary, 8 — slip surface, a — buildings, f — protective fill, r — highway.

13.2.4 Slopes on the margins of brown-coal basins

The margins of brown coal basins are mostly occupied by facilities which have been moved from the centre of the basin, e. g. dwellings, roads, water-mains and even streams. Therefore the relatively steep slopes are required and have to be studied in detail. Also the stability when the pit comes to be filled-in must be considered if the pit is to serve as a hydraulically sluiced ash deposit.

The inclined margin of the coal seam provides a good protective layer for the slope. Despite the poor resistance along the bedding planes, the compression strength of a brown coal seam in the direction of the bedding is relatively high, e. g. of the order of 1.2 MPa. The exact value depends very much on the magnitude of the uplift force which tends to buckle the seam.

The example shown in Fig. 13–7 illustrates a typical coal basin margin. The coal bank (3) remaining on the slope gave way twice after it had been reduced by two successive trial cuts (5 and 6) which were made in order to exploit this part of the coal. The high ground-water table could not be substantially lowered either by two rows

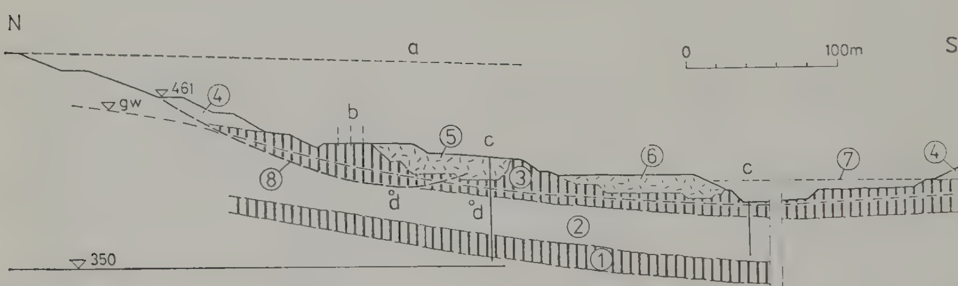


Fig. 13–7. Profile of the margin of an opencast mine near Sokolov; 1, 3 — middle and upper coal seams, 2 — interlayers, 4 — overlying material, 5, 6 — cuttings excavated in the seam and then refilled, 7 — excavation at the bottom of the pit, 8 — slip surface of the first (and most dangerous) of the movements, a — original ground surface, b — platform for mine railway track, c — pumping well, d — horizontal drainage borehole.

of pumping wells or by the horizontal boreholes that were driven from the worked-out area to the east of the profile. The sand inclusions in the interbedded strata (2) prevented proper functioning of the drainage installations. The large uplift force acting at the base of the coal seam contributed to the buckling of the weakened coal bank, although this was still almost 10 m thick. The platform for the mine railway track was seriously damaged. To stabilize the movement, a loading fill was dumped in both of the excavations which had given rise to slope movement (5) and (6). The third cut (7) into the coal seam was successful. A new row of pumping wells was installed and the mining was successfully continued southwards, again leaving a coal bank 7 to 10 m thick at the bottom. Fig. 13–8 shows the filling of the first of the cuttings.

Fig. 13–8. View of slope failure along the slip surface indicated by 8 in Fig. 13-7; a — head scarp, b — direction of movement, c — loading fill.

The final exploitation of an inclined coal seam which has been left as a protective layer on a slope, is not easy. The coal can easily be taken after the track system has been removed, as in the case shown in Fig. 13–7, but in other circumstances permanent facilities of public importance might be endangered, as shown in Figs. 13–9 and 13–10. In the example of Fig. 13–9 the inclined margin of the seam consisted of coal of poor quality which was therefore left on the slope. The horizontal seam was worked as close as possible to the toe and the inclined margin was supported by a fill.

Fig. 13–10 shows a situation similar to that in Fig. 13–9, but here there is a possibility of slope movement developing deep below the bottom of the seam owing to

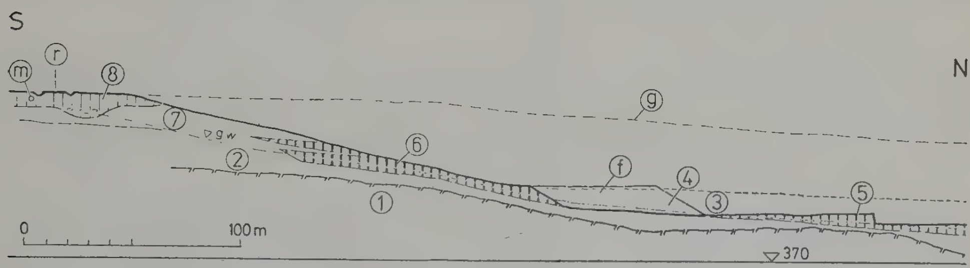


Fig. 13–9. Profile of a slope with an inclined coal seam left as a protective layer (6); 1 — kaolinized gneiss, 2 — clay and sand beds, 3 — seam worked by the standard method, 4 — seam worked by excavating normal to the slope and immediately back-filling (f), 5 — additionally worked coal bank, 6 — lean coal, 7 — overlying clay, 8 — loam and gravel, g — original ground surface, m — water main, r — highway.

the uplift effect of ground-water. The uplift force acts at the surface of contact between the upper, little permeable clayey cover and the more permeable tuffaceous agglomerates underneath.

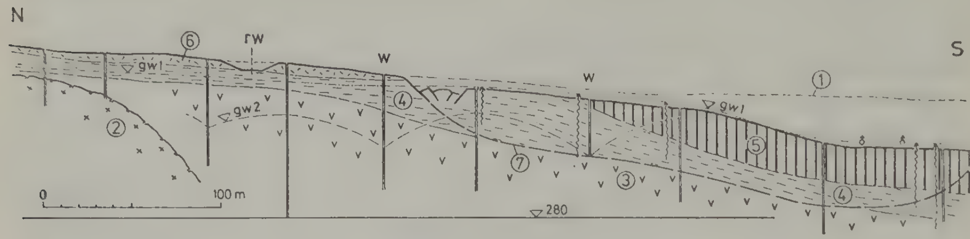


Fig. 13-10. Slope of a brown-coal opencast pit near Chomutov in Bohemia; 1 — original ground surface, 2 — gneiss, 3 — tuff agglomerate, 4 — tuffaceous silty clay, 5 — coal seam, 6 — slope debris, 7 — slip surface, gw 1 — ground-water table before drainage by pumping wells (w), gw 2 — lowered water table, rw — railway line.

13.3 Sliding of pit heaps

(a) The high pit heaps of brown coal mines, consisting mostly of the overlying clay, often contain one or more horizons of water. This water collects on the platforms made during the successive stages of building the heap, and decreases the stability of the heap. Stability is also impaired by softening of the surfaces of the dumped clay lumps. Thus the slope angle necessary for a stable heap may vary substantially. An angle of 9° seems to be reasonable for a heap which is built properly – i. e. without "fingers" to facilitate sliding at the margins, and without the formation of pockets in which water can collect. The advantage of the heap material (with the exception of the old platforms) is its permeability, which allows the heap to be drained easily by means of horizontal boreholes. It has been in pit heaps that the longest boreholes (over 300 m) have been drilled.

(b) Large-scale collapses of heaps occur when the bearing capacity of the substratum is relatively poor; the heaps of coal mines are usually high enough for ductile deformation of the soils at the base to occur. The pressure necessary to induce this behaviour in the hard claystones which usually form the surface layers in brown coal

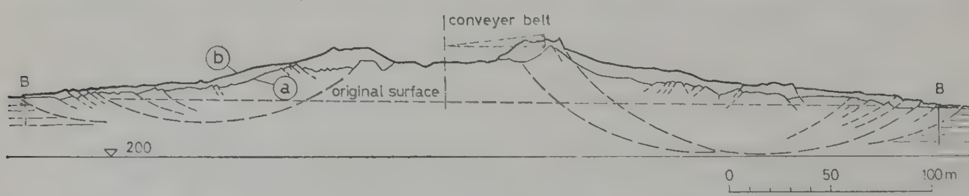


Fig. 13-11. Clay tipped to a temporary waste heap causes squeezing out of the incompetent substratum and sliding of the slopes of the heap (near Most, Bohemia); a— state in May 1948, b — state in August 1948.

basins is about 0.4 to 0.5 MPa. This pressure is equivalent to that which would be produced by a fill 20 to 25 m high. Deep shear zones develop under these conditions and an extensive collapse ensues (Fig. 13 – 11). The profile shown related to conditions prevailing thirty years ago; currently, the stability of further tippings is investigated and dumping is usually carried out in banks. As mentioned earlier, 9° overall slope inclination can be achieved generally. In the case shown in Fig. 13 – 11, the angle reached was 7°.

13.4 Open pits in stable rocks

The authors have had little personal experience with the problems of the stability of deep pits in hard rocks. It will suffice to mention some of the excellent and thorough work which has been done by others in this field of slope problems. The exactness with which the slopes have been designed to collapse just after the closing of the mine is surprising (section 7.2). Careful slope measurements with inclinometers and extensometers (Miller and Hilts 1969), and detailed and systematic laboratory work (Legget 1957) lend firm support to the experience and judgment of the experts. A review of several examples has also been shown by Voight (1979). In principle, the inclination of slopes several hundred metres high does not exceed 45° and is mostly of the order of 38°.

The characteristics of the margins of pits in hard rocks are interesting when compared with those of quarries and other deep excavations. For example, the Neogene Brno sand of the Carpathian Foredeep, which is slightly cemented with carbonates and hydroxides, stands in the form of exactly vertical walls 15 m high. Walls such as this were created in the sand pits several decades ago, and no accidents have been recorded; in several places the abandoned slopes still stand and are stable. However, when road cuttings are excavated in this material a slope inclination of 45° is preferred; Miller and Hilts (1969) described the conditions of a slope 45 m high inclined at this angle in cemented sand overlying a bauxite deposit.

A slope 55 m high inclined at 47° was found by Miller and Wilson to be stable despite excavation of the underlying coal seam. The slope consisted of almost horizontal beds of sandstone, siltstone and claystone of the Upper Eocene, the sandstone being the predominant feature. The mean unconfined compression strength of the sandstone was 0.48 MPa, that of the siltstone was 1.55 MPa, and that of the claystone was 1.06 MPa, the water contents being 17, 17 and 23 per cent, respectively. The friction angles, as determined by laboratory triaxial tests, were 38 to 40° for the sandstone and only 15 to 20° for the claystone, these for the siltstone being intermediate between these ranges. Water was entering the opening cracks and joints behind the slope face and therefore it was not recommended that benches be left on the slope – a course of action which corresponds well with the experience of building railway cuttings in the Carpathians.

Recently, even rock anchors were applied to ensure the stability of a copper mine slope at Tuczon, Arizona (Seegmiller 1979). The slope was more than 400 m high, inclined 45°, in quartzite.

13.5 Quarries and loam-pits

Rockslides, rockfalls and the sliding of large blocks have often occurred in quarries, especially when the standard of geological examination has been inadequate. Incidents such as those shown in Figs. 1–5 and 5–65 probably do not occur under present-day conditions, although both small and large slope movements do take place nowadays. Slope movements in quarries may produce unexpected consequences. Fig. 13–12 shows a quarry for a cement works, where it was found that the irregular mixing of limestone and marl made the extracted material unfit for cement-making. A new quarry had to be opened on the south flank of the hill.

The sliding of loam-pit slopes is a frequent occurrence, especially where precautions are relaxed because there is no danger to property or facilities on the inside or the outside of the pit. Sometimes, however, movements assume larger dimensions than

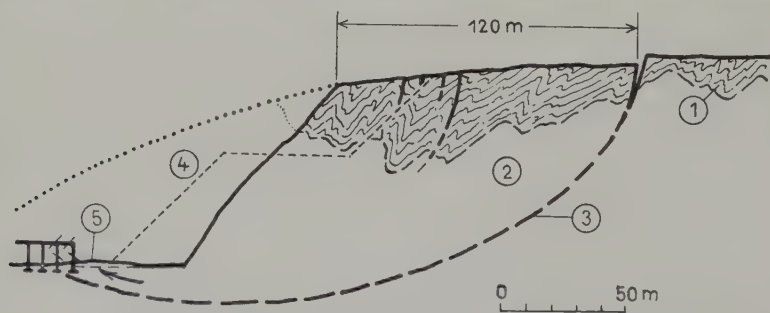


Fig. 13–12. The steepness of the quarry face disturbed the stability of the entire slope in this quarry near Trenčín in Slovakia; 1 — Jurassic limestone, 2 — Cretaceous marls, 3 — presumed slip surface, 4 — loading fill, 5 — squeezed-up marl which damaged the quarry plant (Záruba and Mencl 1954).

anticipated, and begin to threaten neighbouring structures standing near the toe of the slope or behind its crest. In the former case the yard managers may try to stop the movement by surface drainage, since the economics of running the quarry does not allow for more expensive measures. Valuable recommendations are unfortunately often neglected as for example to install drains in the forefront of the moving slope, although they are not costly and are capable of stabilizing the displaced mass.

Those structures standing near the pit are in greatest danger. Fig. 1–8 shows an example of the great damage that can be caused, and yet could be prevented if short-term economic interests do not determine the manner in which the pit is operated.

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SUBJECT INDEX

tion of computing techniques have led to much progress with respect to theoretical analyses and data handling. Chapter 2 (Factors causing mass movements) has also been extended by a new section (Landslides in seismic regions) which gives information particularly about some of the largest seismically induced landslides. The new edition contains four new chapters which deal with slope stability problems in different branches of the building and mining industries, i. e. in dam construction (Chapter 10), in road construction (Chapter 11), in urban planning (Chapter 12), and in the exploitation of mineral deposits, particularly in opencast coal mining (Chapter 13).

The book is a rich source of information for those engineers and engineering geologists whose task it is to recognize and avoid slopes threatened by sliding and to control or correct slope failures. The solution of present development problems demands good scientific knowledge and great engineering skill, some of which we hope may be found in this publication.

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ISBN 0-444-99700-8