CANADA

DEPARTMENT OF
ENERGY, MINES AND RESOURCES
MINES BRANCH
OTTAWA

PLANNING SLOPES IN SHALE AND OTHER ROCKS

D. F. COATES

FUELS AND MINING PRACTICE DIVISION

Conference Preprint 342 of ASCE Water Resources

Engineering Conference/Denver, Colorado, May 16-20, 1966

Reprint Series RS 27

Price 25 cents

Available by mail from the Queen's Printer, Ottawa,
and at the following Canadian Government bookshops:

OTTAWA
Daly Building, Corner Mackenzie and Rideau

TORONTO
Mackenzie Building, 36 Adelaide St. East

MONTREAL
Aeterna-Vie Building, 1182 St. Catherine St. West

or through your bookseller

A deposit copy of this publication is also available
for reference in public libraries across Canada

Price 25 cents   Cat. No. M38-8/27

Price subject to change without notice

ROGER DUHAMEL, F.R.S.C.
Queen's Printer and Controller of Stationery
Ottawa, Canada
1966

PLANNING SLOPES IN SHALE AND OTHER ROCKS

by

D.F.Coates[1]

Synopsis

In rock slopes it is difficult to follow the traditional design procedures as there are no theoretical analyses that can be used for the determination of stresses. Only in the cases where rocks yield sufficiently to be analogous to soil can average stresses be used as implied by the slip circle analysis. Furthermore, with failure being a stochastic phenomenon (unlike the traditional assumption in soil mechanics where variability is ignored) and with the strength of rock samples being known to vary inversely with their volume, the prediction of effective strength in a rock slope will not be easy.

Of the various types of rock slope failures - rockfalls, rotational shear, plane shear and block flow - the possibility of the latter two occurring is generally of most concern to those planning excavated slopes. The difficulty in predicting a plane shear failure is in locating any critical extensive planes of weakness. The difficulty in predicting block flow failure is that the mechanics involved are not understood.

In reviewing the properties of shales, it is found that they vary through the complete spectrum of rock properties from weak, yielding materials to strong, elastic, brittle materials. Viewed in this light it can be argued that a functional classification of the rock at the site would

See over

---

[1] Head, Mining Research Laboratories, Department of Mines and Technical Surveys, Ottawa, Canada

be more useful than a name based on genesis or composition. The peculiar properties of some shales of decrepitation and swelling on exposure might be considered simply as reactions affecting the long-term strength of the material. The case histories reviewed in the paper tend to support this orientation.

At the present time, it is suggested that for the planning of slopes in shales and other rocks, the practical procedure is to concentrate on monitoring deformation rather than analysing stress. In fact, it is thought that it may eventually be proved that deformation is a better parameter than stress for predicting and analysing the reaction of rock to both surface and underground excavations.

INTRODUCTION

In initiating this paper the suggestion was made that it should be on the design of slopes in shale. As the design of slopes in any rock at the present time is very difficult, the paper has been written on the planning of the slopes. The word 'planning' is to be interpreted in the more general sense of 'considerations to be aware of in planning' rather than being a definite procedure or check list for practical purposes.

The subject is treated under the headings of: design - types of slope failure - classification of shales - case histories - and planning.

SLOPE DESIGN

Conventional Procedure

Structural design is usually conducted by a traditional cut-and-try procedure. A preliminary design is established, the stress in the materials are analysed, and the strengths of the materials are established by laboratory testing of representative samples. When a design produces an acceptable safety factor with respect to failure stresses, the basic configuration can be accepted.

In the case of slopes in soil, this traditional procedure can be followed if the assumptions are made that failure will occur on a circular surface of failure and that the average shear stress along this surface is the critical stress to be analysed for design purposes. These assumptions are only likely to be valid in materials where the variations in stress which tend to occur will be diminished, or eliminated, by local yielding without fracture in zones of high stress with resultant transfer and increase of stress in the adjacent zones.

Stresses in Infinite Slopes

In rock slopes it is difficult to follow the traditional design procedure for several reasons. First, there is at the present time no theoretical analysis that can be used for the determination of the stresses in an elastic mass (the normal first assumption for the material properties) with a geometry of a slope. If the complications of having a crest and toe are eliminated, in other words if the case of an infinite slope is assumed, stresses can be analysed as indicated in Figure 1 producing the following equations:

$$N = W \cos i = \gamma z (b \cos i) \cos i$$

$$T = W \sin i = \gamma z (b \cos i) \sin i$$

$$\therefore \quad \sigma = \gamma z \cos^2 i \qquad \text{..... (1a)}$$

$$\tau = \gamma z \sin i \cos i \qquad \text{..... (1b)}$$

where N is the normal reaction on the bottom of a vertical column of ground, W is the weight of the column, T is the tangential reaction on the bottom of the column, is the density of the ground, z is the depth of the column measured vertically, b is the width of the column measured parallel to the slope, i is the slope angle, σ is the normal stress on the bottom of the

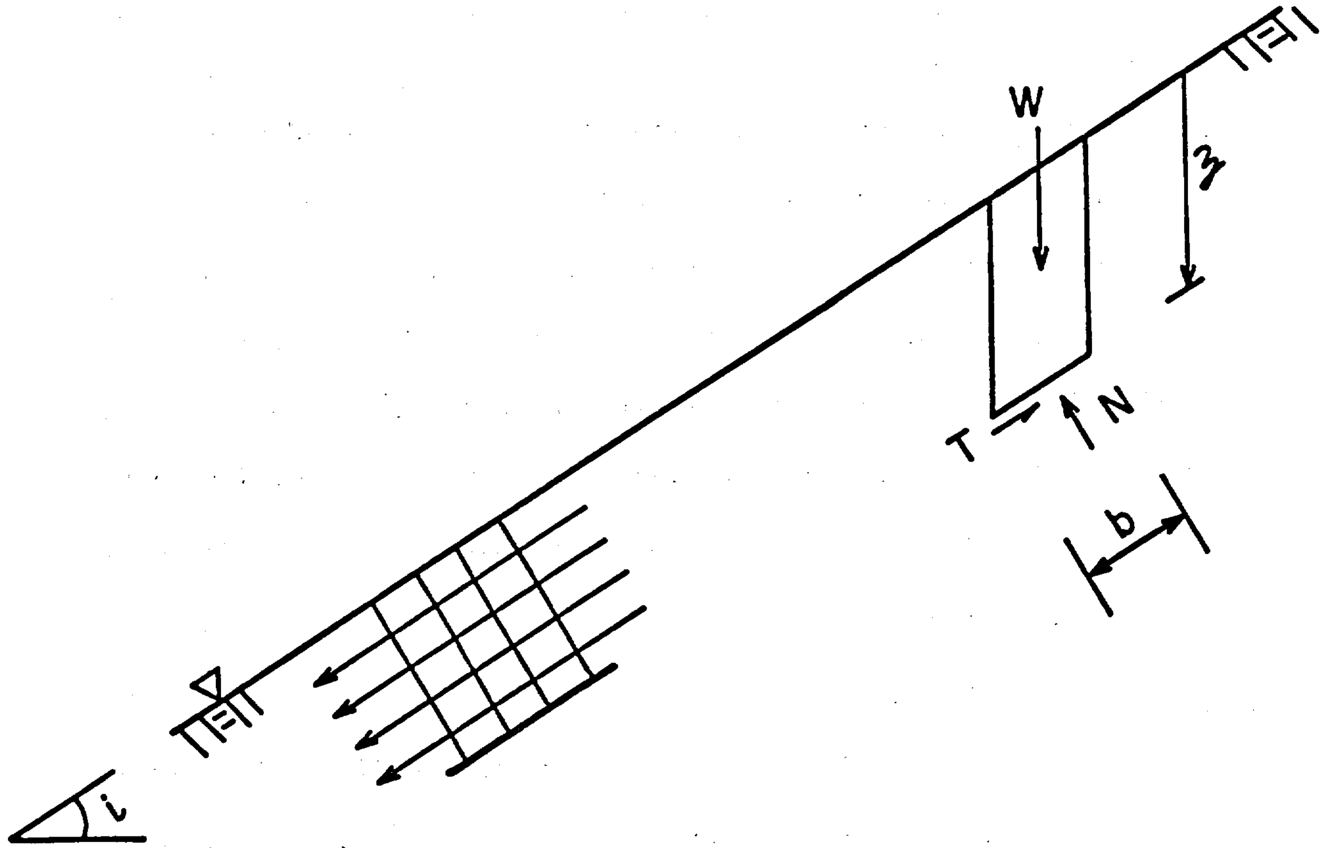


FIGURE 1 - AN INFINITE SLOPE WITH AND WITHOUT SEEPAGE

column and τ is the shear stress. The vertical column of ground isolated in Figure 1 will be subjected to forces on the sides of the column that are equal and opposite: for this reason, they do not appear in the equations.

If seepage is occurring in the infinite slope, as shown in Figure 1, with flow parallel to the surface and the phreatic line coincides with the surface of the slope, the additional components of stress are as follows:

$$u = \gamma_w z \cos^2 i \qquad \text{..... (2a)}$$

$$J = \gamma_w \sin i \qquad \text{..... (2b)}$$

where u is the pore pressure at the depth z below the surface of the slope, γ_w is the density of water and J is the seepage stress per unit volume of ground (note that the hydraulic gradient in this case equals sin i).

Stresses in a Semi-Infinite Slope

The slope with a crest but no toe, or a semi-infinite slope, as shown in Figure 2a, can be taken as the next degree of complexity. Equations for the determination of stresses in such a slope have been published (1,2)+:

$$\sigma_r = \gamma r \left[\left(\frac{\cos 3\, i/2}{8 \sin^2 i/2} + \cos i/2 \right) \cos \psi + \left(\sin i/2 - \frac{\sin 3\, i/2}{8 \cos^2 i/2} \right) \sin \psi - \frac{\cos i/2 \cos 3\psi}{8 \sin^2 i/2} + \frac{\sin i/2 \sin 3}{8 \cos^2 i/2} \right] \qquad \text{..... (3a)}$$

$$\sigma_t = \gamma r \left[\left(\frac{3 \cos 3\, i/2}{8 \sin^2 i/2} + \cos i/2 \right) \cos \psi + \left(\sin i/2 - \frac{3 \sin 3\, i/2}{8 \cos^2 i/2} \right) \sin \psi + \frac{\cos i/2 \cos 3\psi}{8 \sin^2 i/2} - \frac{\sin i/2 \sin 3\psi}{8 \cos^2 i/2} \right] \qquad \text{..... (3b)}$$

$$\tau_{rt} = \gamma r \left[\frac{\cos 3\, i/2 \sin \psi + \cos i/2 \sin 3\psi}{8 \sin^2 i/2} + \frac{\sin 3\, i/2 \cos \psi + \sin i/2 \cos 3\psi}{8 \cos^2 i/2} \right] \qquad \text{..... (3c)}$$

+Numerals in parenthesis - thus; (1) - refer to corresponding items in the Reading References (see Appendix).

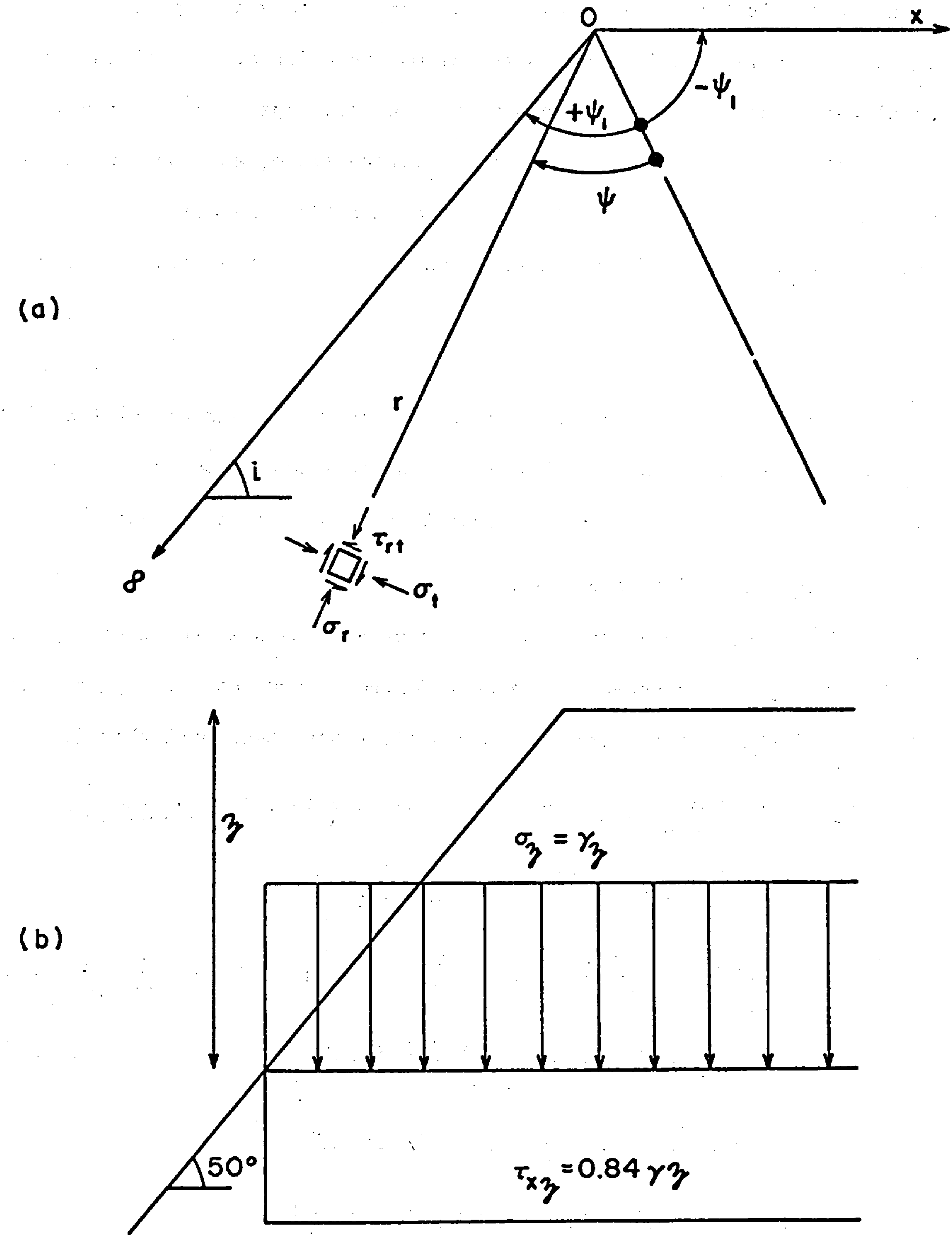


FIGURE 2 - A SEMI-INFINITE SLOPE : a) the stress co-ordinates, and b) a graph of shear stress and vertical stress on a horizontal plane.

where σ_r is the radial stress from the origin O at the crest, γ is the density of the ground, r is the radial distance from the origin, i is the slope angle, ψ is the angle measured clockwise from the bisector of the interior angle to the crest, σ_t is the tangential stress and τ_{rt} the shear stress on radial and tangential planes.

Presumably these equations could be used for points sufficiently above the toe not to be influenced by the stress concentrations there. For example, with a 50 degree slope, it was found, as shown in Figure 2b, that the shear stress on horizontal planes is 0.84 γz, where z is the depth to the plane, and that this value is constant for all values of x. Also Figure 2 shows that the vertical stress, σ_z, is equal to γz.

The basic assumption for Equations 3 is that σ_r, σ_t and τ_{rt} all vary linearly with the radial distance r. This may or may not be true; the result that the horizontal stress, σ_x, increases indefinitely with x suggests that this derivation is not a valid solution. Furthermore, the assumptions and deductions regarding the variations of the various stresses do not seem to be in agreement with recent experimental work; Figure 3 shows the results obtained on a gelatin model (3). Although this was not a case of a semi-infinite slope, sections A and possibly B should be far enough from the toe not to be influenced by the stress concentrations there. The vertical stress, σ_z , and the shear stress, τ_{xz} , are not constant at one elevation as indicated by the above theory. Also, the above equations do not indicate the presence of tensile stresses in the crest zone, which are known to exist in finite slopes, although they may not be produced in semi-infinite slopes. At section C in Figure 3, the effects of the concentration of stress at the toe can be seen.

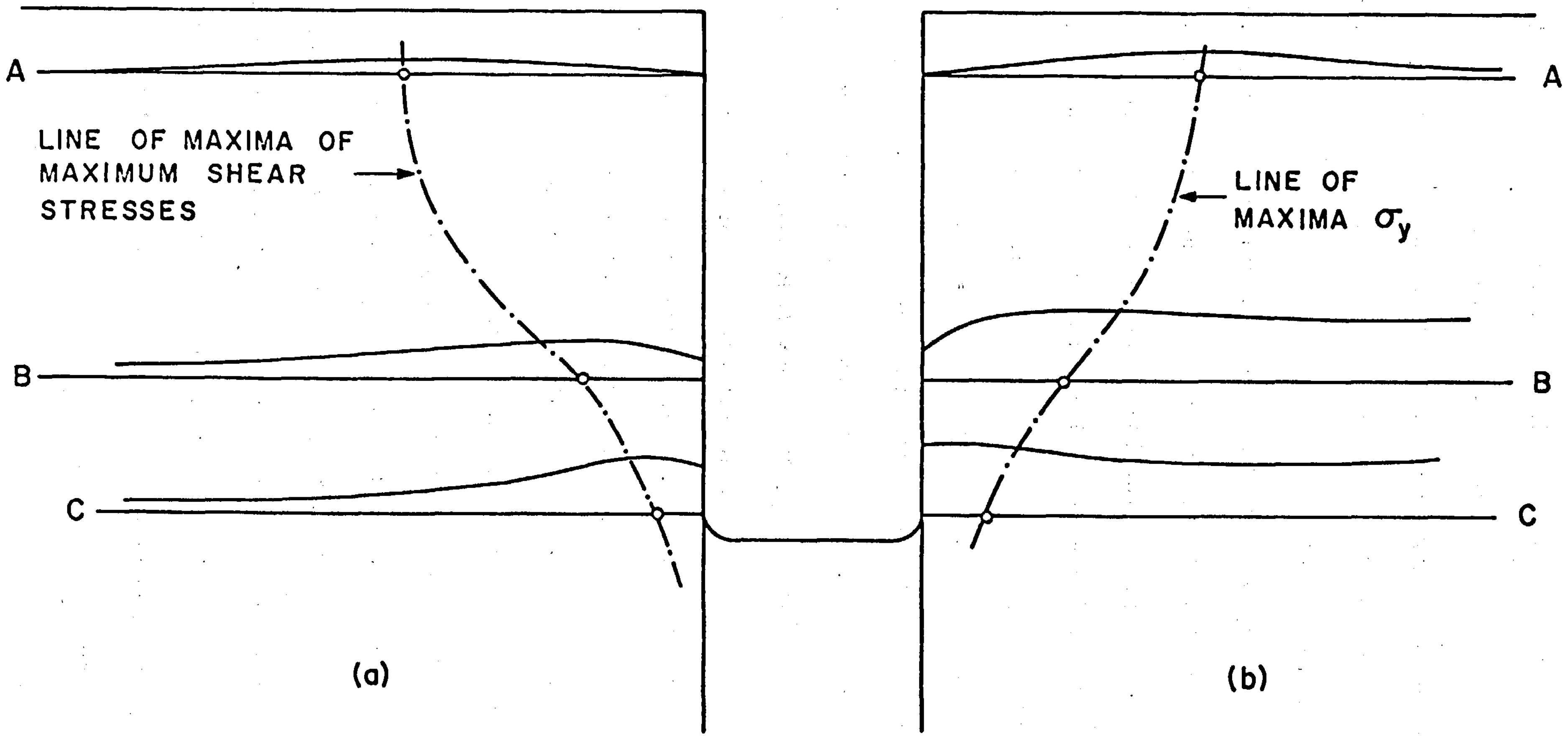


FIGURE 3 - GELATIN MODEL EXPERIMENTS SHOWING THE VERTICAL STRESSES AND SHEAR STRESSES ON HORIZONTAL PLANES THROUGH VERTICAL SLOPES

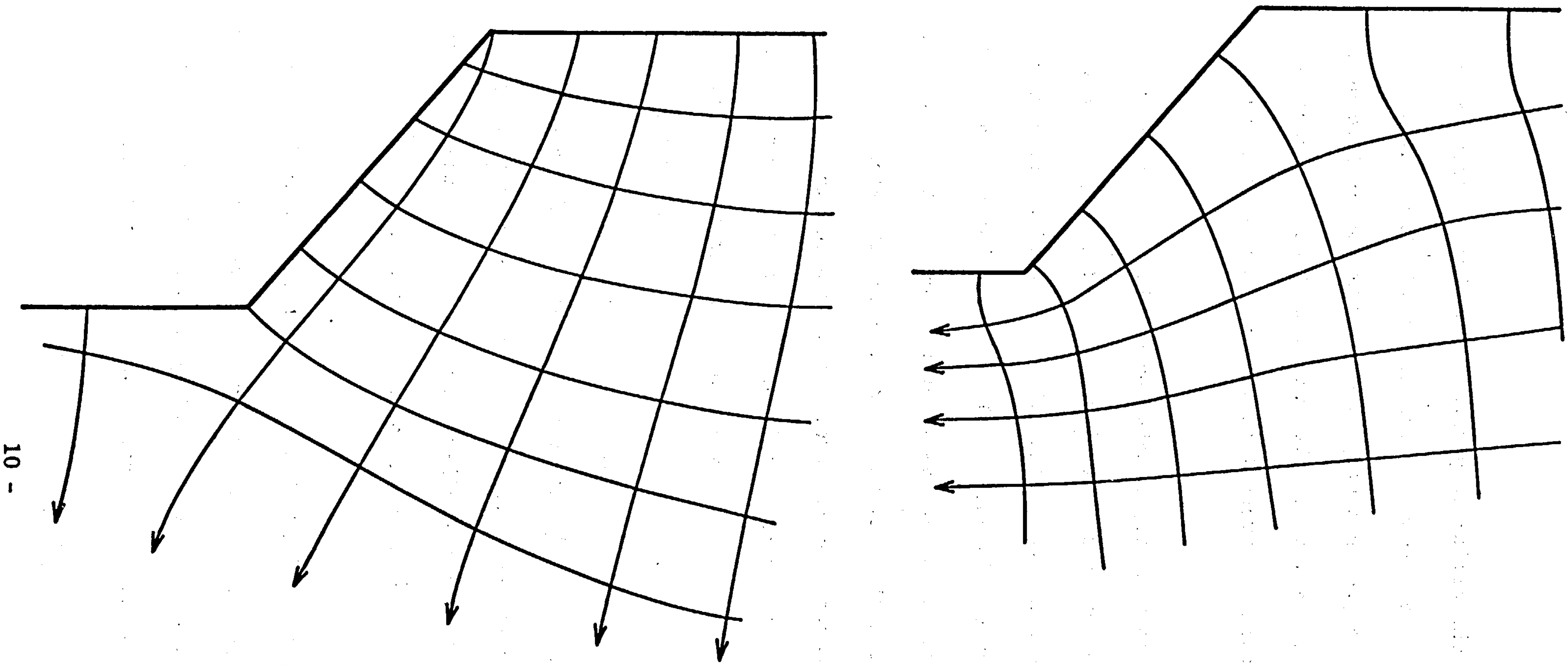
Stresses in Finite Slopes

It is possible that the difficult problem of analysing stresses in slopes may be solved in the same manner as the mathematically similar problem of fluid flow with complex geometry, i.e. by the use of flow nets. The principal stress trajectories, as in flow nets, must be orthogonal and with the appropriate selection of stress intervals the elements of the mesh can have equal sides.

Figure 4a is an approximate representation of the principal stress trajectories where the major principal field stress is due to gravity and hence vertical. Figure 4b shows the principal stress trajectories where the major principal field stress is horizontal. In the latter case, it is quite clear that the maximum normal stress is at the toe of the slope and that there is a concentration of horizontal stress under the excavation beyond the toe. (This concentration of stress is also shown in Figure 3 at section C.) This method, of course, could not provide a measure of the theoretical maximum stress concentration which would occur at the surface of the slope at the toe; it remains for research to establish whether this is a critical number with respect to a design analysis.

Alternatively, stress distribution can be determined for a given slope using the finite element technique based on the use of a high speed computer. It is possible that the availability of computers will shortly be so widespread that such an approach can be used for routine design purposes.

Stresses in slopes, as implied above, will not always be due simply to the force of gravity. As insitu measurements in rock are continued, it is being found that horizontal stresses are often greater than the vertical stresses and that only in special cases can it be assumed that the major



(a) (b)

FIGURE 4 - FLOW NETS OF PRINCIPAL STRESS TRAJECTORIES a) where the major principal field stress is vertical and b) where the major principal field stress is horizontal.

principal stress in the undisturbed rock is vertical and due to gravity, with the intermediate and minor principal stresses being some fraction of this due to lateral confinement of the ground. The case has been documented where an open pit excavation had proceeded no more than 50 feet below the ground surface when the floor cracked and heaved up some 8 feet, a clear indication of large residual stresses in this near-surface bedrock (4).

Strength of Rock Masses

The other aspect of design, the testing of representative samples for the determination of the strength of the material, is in an equally unsatisfactory state of development. Tests on core samples at best (and even the laboratory testing of brittle materials is not completely satisfactory at the present time) merely provide a measure of the strength of the substance, which in most cases bears no predictable relationship to the strength of the rock mass. The strength of the substance can be considered to provide an upper limit to the strength of the rock mass, but even as such it usually has limited value.

The strength of a rock mass in a slope is probably affected by the orientation and frequency of joints. The determination of the orientation of the various systems of joints must usually be done on a statistical basis, thus requiring a large number of observations. The observations can be obtained by measurements on the joint faces exposed either in outcrop or in excavations in the same formation and from observations in drill holes using either a borehole camera or a closed circuit TV camera.

These techniques are expensive and not perfect from a scientific point of view. First, the identification of a joint is not completely unequivocal (5). Then, observations of frequency on the face of an excavation are biased towards those joints in planes perpendicular to the face. Borehole observations are somewhat better, but frequency is biased against orientations parallel to the axis of the hole and detection by camera is biased against those normal to the hole, i.e. their trace is less conspicuous than joints that approach being parallel to the axis of the hole.

At the present time, the proper use of such joint data in an analysis of the mechanics of the slope is still a matter for theoretical consideration and thus is still in the realm of research rather than practical engineering.

At Mines Branch the problem of determining the mechanical properties of rock masses is being approached from several different directions. Stress measurements are being made in stable and unstable underground pillars which provide a measure of the strength of the rock mass in the pillar. Extensometer readings together with these stress measurements are used to determine deformation properties. Sonic velocity measurements are being made in connection with such stress measurements to assist in delineating stable from fractured ground (6). A borehole penetrometer is being used in an attempt to build up a correlation between hardness readings in a borehole and the strength of the adjacent core so that such a correlation might be used to extrapolate to rock where core cannot be obtained but a hardness measurement can (7). Plate load testing has been tried but is not now considered to be a particularly feasible method for obtaining the strength of the rock mass (8). The Schmidt impact hammer is being tried for the determination of at least comparative competancy. Trials are to be conducted on a borehole cell, or dilatometer (9). However, it will take time before any of these methods are substantiated.

The final difficulty in following the traditional design analysis in slopes is that even if we could make a stress analysis and determine the strength of the rock, it has not yet been established what the failure criterion is for brittle rocks nor what the mechanics of failure are for a slope not affected by some obvious structural features. Concrete work is essentially based on the Maximum Principal Stress Theory for compressive stresses and the Maximum Shear Stress Theory for shear stresses; soil work uses Mohr's Strength Theory, and testing on glass indicates that Griffith's Theory is valid. Recent work now suggests that a combination of Mohr's theory with Griffith's Theory, in order to include frictional resistance on the surfaces of cracks that have been closed, might be the most valid theory for brittle rocks (10). All of these theories may have some validity depending on the rock types; however, it will take time to clarify this subject. On the other hand, if the lack of a strength theory were the only impediment to designing slopes the use of engineering judgment would permit some design procedure to be established, as it has done for many years in concrete work.

TYPES OF SLOPE FAILURES

The four general types of failure, as shown in Figure 5, are distinguished by the mechanics of the breakdown of the ground.

Rock Falls.

Rock falls occur when loose rock develops with the slope angle being greater than the angle of repose of this loose material and blocks of rock fall or roll to the bottom of the slope. The cause of this type of slope failure is the breakdown of the tensile strength of the rock mass.

Rotational Shear

Rotational shear failure occurs when a segment of the slope fails by

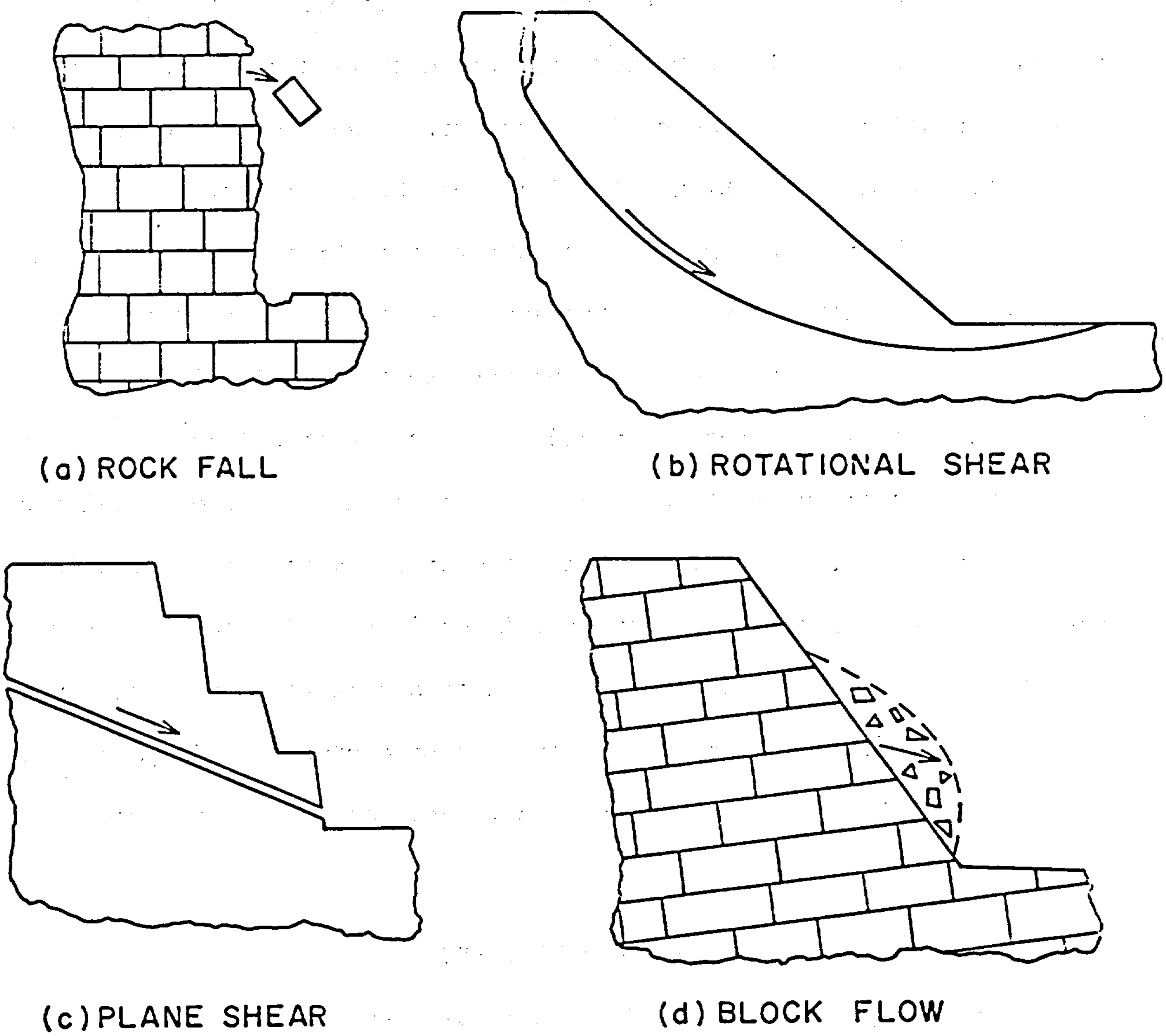


FIGURE 5 - TYPES OF SLOPE FAILURE : a) rock falls, b) rotational shear where yielding prevents high stress concentrations, c) plane shear where an extensive geological plane of weakness exists in a critical location, and d) block flow where a hardrock mass does not contain extensive planes of weakness oriented in a critical direction; the brittle blocks crush at points of load concentration and eventually a general breakdown occurs.

rotation on a more or less circular arch. This type of failure only occurs in rocks that will yield, in a manner similar to structural steel or soils, before rupturing. This yielding is necessary so that high local stresses are prevented from developing. Rupture only occurs when the average shear stress over a critical surface exceeds the average shear strength.

Plane Shear

Plane shear describes the slope failure when sliding occurs along an extensive geological plane of weakness. Failure results from the critical location of such planes with respect to the geometry of the wall.

Block Flow

This type of failure is imagined to occur in a uniform, hard rock mass that does not contain extensive planes of weakness oriented in a critical direction. The method of failure is visualised as being a general breakdown of the rock mass as a consequence of the crushing at the points of high stress in the brittle blocks which comprise the mass. As individual blocks are crushed, increased loads are thrown onto the adjacent blocks whose strength in turn is exceeded. It is thought that this type of progressive action, or working of the ground, continues until a general breakdown of the slope occurs.

CLASSIFICATION OF SHALES

Geological and Structural Aspects

Shale has been defined as a laminated sediment in which the constituent particles are predominantly of clay grade (11). Presumably then the constituent minerals are those such as kaolinite, illite and montmorillonite.

Possibly the more pertinent definition that is required is that of a rock. It has been suggested that the essential structural nature of a rock is that it is a material composed of blocks joined with a cement of weaker strength than the blocks and with the assemblage having a void ratio close to zero based on the density or specific gravity of the substance in the blocks (12). For one limiting case where the strength of the cement is equal to that of the blocks, we would have the equivalent of a cohesive soil. Another limiting case would be when the void ratio is greater than some limiting figure, say 0.05 or possibly some higher limit, a granular mass or soil would exist (13, 14). Figure 6 shows the drastic fall in E, the modulus of deformation, when e, the void ratio, exceeds 5% for this particular rock (14). With some such definition it would be possible to establish a boundary line between shales and clay on the one hand and shales and either breccias or residual soils on the other.

Functional Aspects

The fact that shales are sedimentary and composed of clay minerals may or may not be significant for those interested in the mechanical aspects. If adhesion along the bedding planes is low then bedding would be significant; on the other hand, if the bedding is difficult to distinguish it would be irrelevant. Extensive insitu shear testing of several different shales has shown little difference in strength parameters (C and ϕ) for shearing parallel and perpendicular to the bedding (13).

If because of composition a shale will decrepitate on exposure or swell on release of stress, as might be expected in the presence of montmorillonite but not with other minerals, then composition would be significant; otherwise it would not. In other words, mechanical rather than genetic considerations should be emphasized.

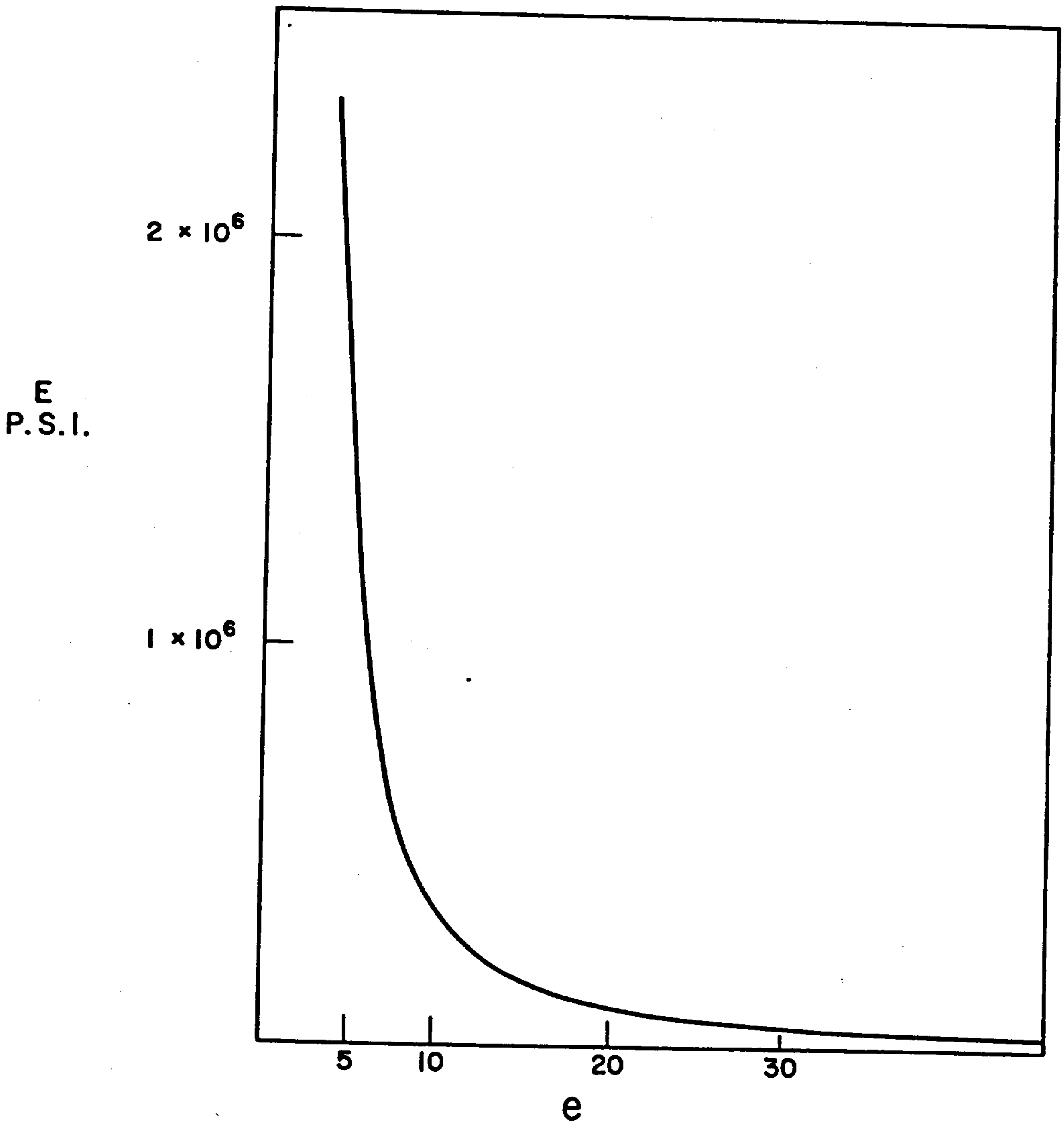


FIGURE 6 - TYPICAL VARIATION OF E , THE MODULUS OF DEFORMATION, WITH e , THE VOID RATIO, SUGGESTING A CRITERION BASED ON VOID RATIO TO DISTINGUISH BETWEEN ROCKS AND SOILS

With this in mind a rock classification system has been suggested for consideration (15). This system is simple to the point of being crude and is visualized as an initial description, with elaborations grafted on as they are established as being functional. Detailed information is regarded as engineering data which is only needed for a particular project.

A modification of the categories originally suggested for classification is as follows:

Substance: Strong or Weak

Elastic or Yielding

Formation: Massive, Layered, Blocky, Broken .

The strength of the rock substance could be defined as strong when the uniaxial compression strength is greater than 10,000 psi. Below this value the rock substance would be described as weak. In view of the limited use for analytical purposes that can be made of the strength of the substance, the two divisions should be adequate for classification. Where useful the system could be expanded to form additional categories by calling rock substances very strong when their compressive strengths are greater than 25,000 psi and similarly describing as very weak those rock substances with compressive strengths less than 5,000 psi.

The classification of the deformation properties of the rock substance should provide some information on the possibilities of yielding, creep, swelling, dissipation of strain energy and nature of failure. An elastic rock could be expected not to creep or swell; it would also be expected to store a relatively large amount of the strain energy so that on failure, which would probably be by brittle fracture, this energy would be released explosively. The term yielding could be used to describe those rock substances where more than 25% of the total strain is irrecoverable or the strain rate at constant stress is greater than 2 μin./in./hr. Among these materials some would creep, some would swell on exposure, some would fail by yielding rather than rupturing and even those that ruptured would not do so with the explosive violence associated with elastic rocks. For slope work these materials might coincide with the yielding required to produce a rotational shear failure as opposed to block flow failure in the brittle rocks.

It should be appreciated that these categories for the rock substance only provide one boundary for the properties of the rock mass. In other words, if a rock substance is strong, the rock mass might also be strong or it might be weak due to structural features. On the other hand, if the rock substance is weak then certainly the rock mass will be weak. Similarly, if the rock substance is of a yielding type, the rock mass will also be a yielding body.

The continuity of the formation can be described as massive when the distances between layers and joints are more than 6 feet. The formation would be layered when the rock is either sedimentary, extrusive in distinct layers, or in compositional bands of gneisses and schists with the distance between the layers being less than 6 feet. Layering in a mechanics sense should mean that the bonding between layers is less than it is within any one layer.

The structure of the rock mass can be described as blocky when the joint spacing is greater than 1 foot and less than 6 feet, and as broken when the individual blocks are smaller than 1 foot.

With respect to this classification system, we know that shales can have substances which vary between very strong and very weak, elastic and yielding. The structural characteristics can vary between massive, layered and broken. The logical conclusion from this is that shales are a compositional grouping of rocks which can have mechanical properties that cover the full spectrum of rock properties.

Figure 7 shows stress-strain curves for a very strong, elastic, massive shale. This shale is from the Lower Cretaceous Elk River formation of British Columbia. It is very fine-grained, black and with no cleavage or bedding planes visible, and hence it might be objected that it is a mudstone rather than a shale. These curves show that the uniaxial compressive strength, Q_u, is greater than 25,000 psi and hence can be classified as very strong.

The modulus of deformation obtained on the loading cycle up to 50% Q_u varied between 5.5 and 7.9 x 10^6 psi for six samples. The air dry density of the rock substance was 164 pcf. The test samples were 1.21 in. diameter with a length to diameter ratio of 2.57 .

The curves in Figure 7 show the permanent strains in the rock substance obtained by cycling the stress to 25%, 50% and 75% of the ultimate compressive strength. The test was carried out as rapidly as the manual controls would permit to eliminate any creep effects. The extrapolated permanent strain at failure would be about 7% of the total strain.

The inset on Figure 7 shows a strain-time curve resulting from a different type of test. In this case the loading and unloading cycles were conducted at the same rate as in the previous test but the stresses at the levels at 25%, 50% and 75% Q_u were maintained constant for 30 minutes with strain readings being taken during this time interval. By plotting this data on a log-log graph and extrapolating the strain rate to a time of

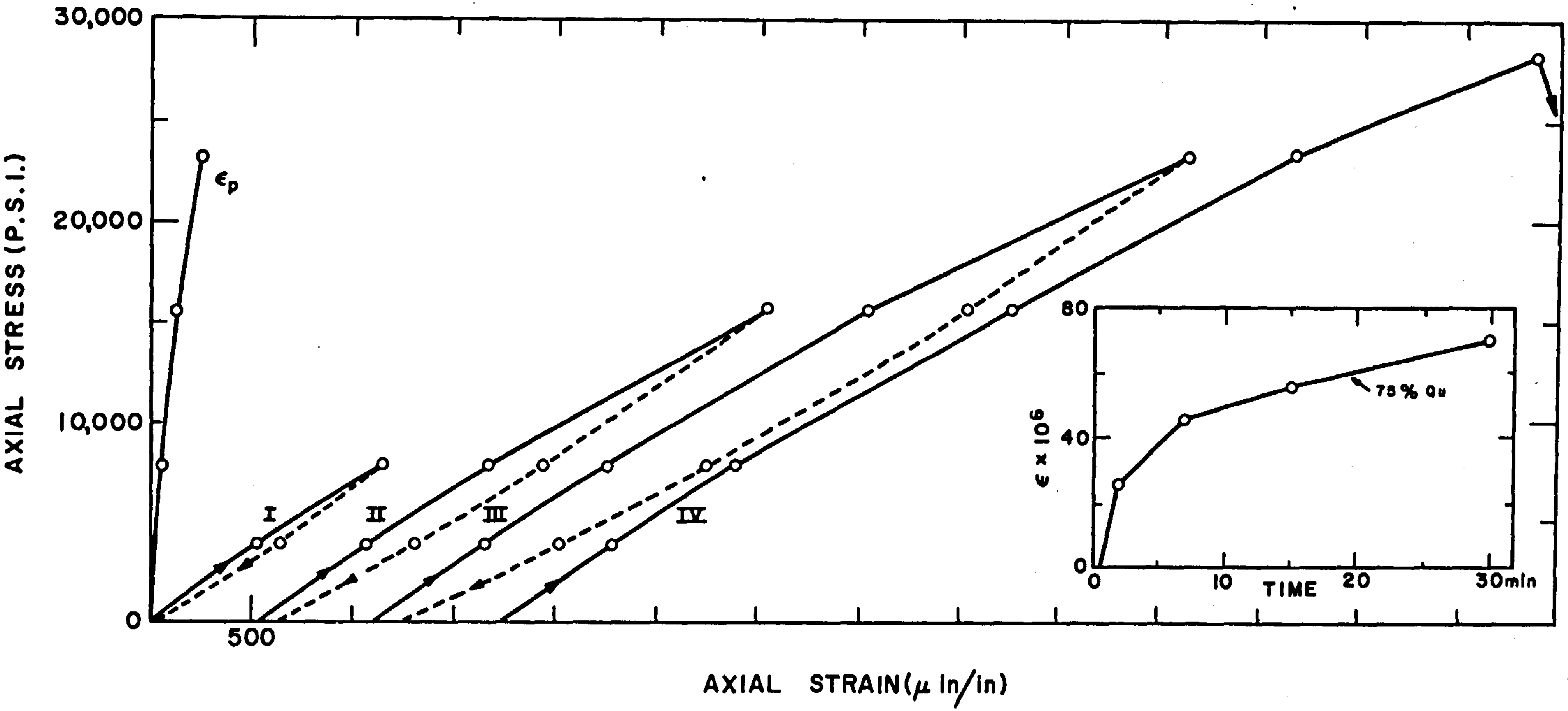


FIGURE 7 - ELASTIC, PLASTIC AND VISCOUS STRAIN VARIATION WITH STRESS IN A VERY STRONG SHALE

200 minutes, assuming that a steady state creep would then be attained (16) it was found that the creep rate would be 1.8 μin./in./hr at a stress level of 50% Q_u. It is judged, based on a rate of closure in an underground opening that would obviously be described as creep, that the classification <u>yielding</u> should apply to rocks with a steady state strain rate greater than 2 μin./in./hr. Accordingly, this shale would be described as <u>elastic</u>.

Figure 8 gives stress-strain and strain-time curves for a sample from the Bearpaw formation, a <u>very weak</u>, <u>yielding</u> Upper Cretaceous shale (17). This material is at the other end of the scale of rock properties, and in fact may be off the scale as it has a void ratio of about 0.28, which together with its cohesiveness would, according to the above definition of rock, place it in the category of a cohesive soil. In any event, it is a quite different material from the Elk River shale.

CASE HISTORIES OF ROCK SLIDES

<u>Rotational Shear A</u>

Nine slides have been recorded in a Keewatin sedimentary rock composed of some clay minerals, known as paint rock, which contains kaolin, quartz and pyrolusite with some sub-angular fragments of iron oxides. The formation is composed of laminations less than 1 cm thick dipping at approximately 70 degrees. It could be classified as <u>very weak</u>, <u>yielding</u> and <u>massive</u>. However, it is questionable whether it should be called a rock as the natural void ratios are approximately 0.4 .

Table 1 shows the data for these nine slides. Laboratory tests established the angle of internal friction to be 37 degrees. Using this figure the cohesion at failure was calculated for the slide geometries assuming the ground water level behind the slope to be at an elevation equal to half the height of a slope (except for Slide No.7, assumed to be full height)

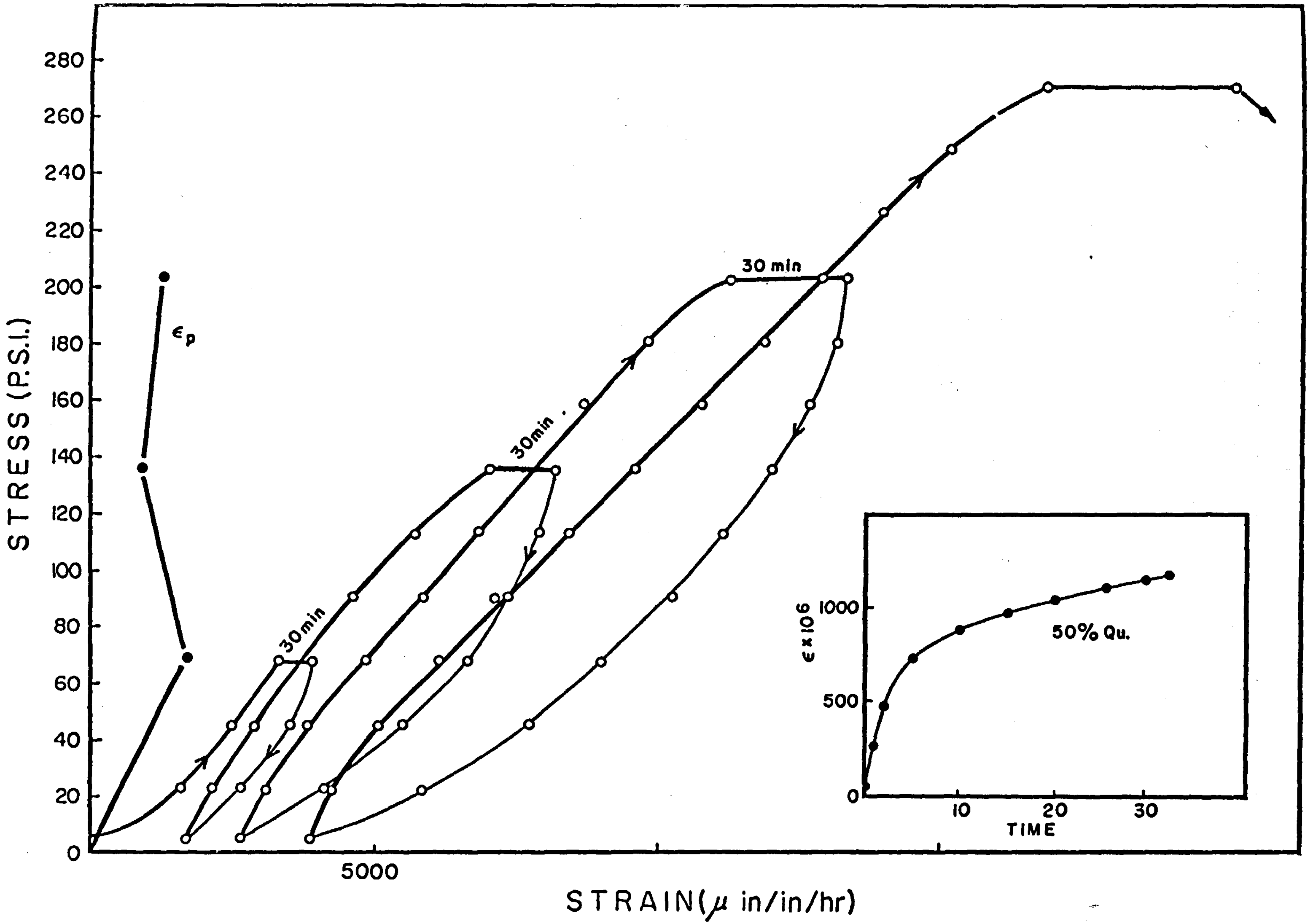


FIGURE 8 - ELASTIC, PLASTIC AND VISCOUS STRAIN VARIATION WITH STRESS IN A VERY WEAK SHALE

Table 1. - Paint Rock Slides

Slide No.	Height, Ft	Width, Ft	Slope Angle, Degrees	Cohesion, Psf
1	215	290	51	1490
2	163	240	51	1150
3	51	300	60	685
4	173	120	50	935
5	138	125	56	1250
6	119	100	54	950
7	86	150	49	660
8	115	50	57	825
9	95	100	66	1195

and assuming the presence of tension cracks equal to half the height of the slope. The average cohesion obtained from these calculations is 1,000 psf with a coefficient of variation of 22%. From this it follows that one-sixth of the ground would have cohesions less than 780 psf. Figure 9 is a photograph of one of these slides.

Block Flow

An example of what was probably a block flow failure occurred in a slope made up of Paleozoic limestone and Cretaceous shales, sandstone and coal. This is the Frank slide where 80,000,000 tons of rock flowed as a fluidized mass 2½ miles in less than 100 seconds (18). The geometry of the slope is shown in Figure 10. The maximum slope angle was 53 degrees with a height of 2100 feet. The bedding dipped 49 degrees into the slope and contained a well-developed family of joints.

Several factors combined either to increase the shear stresses or

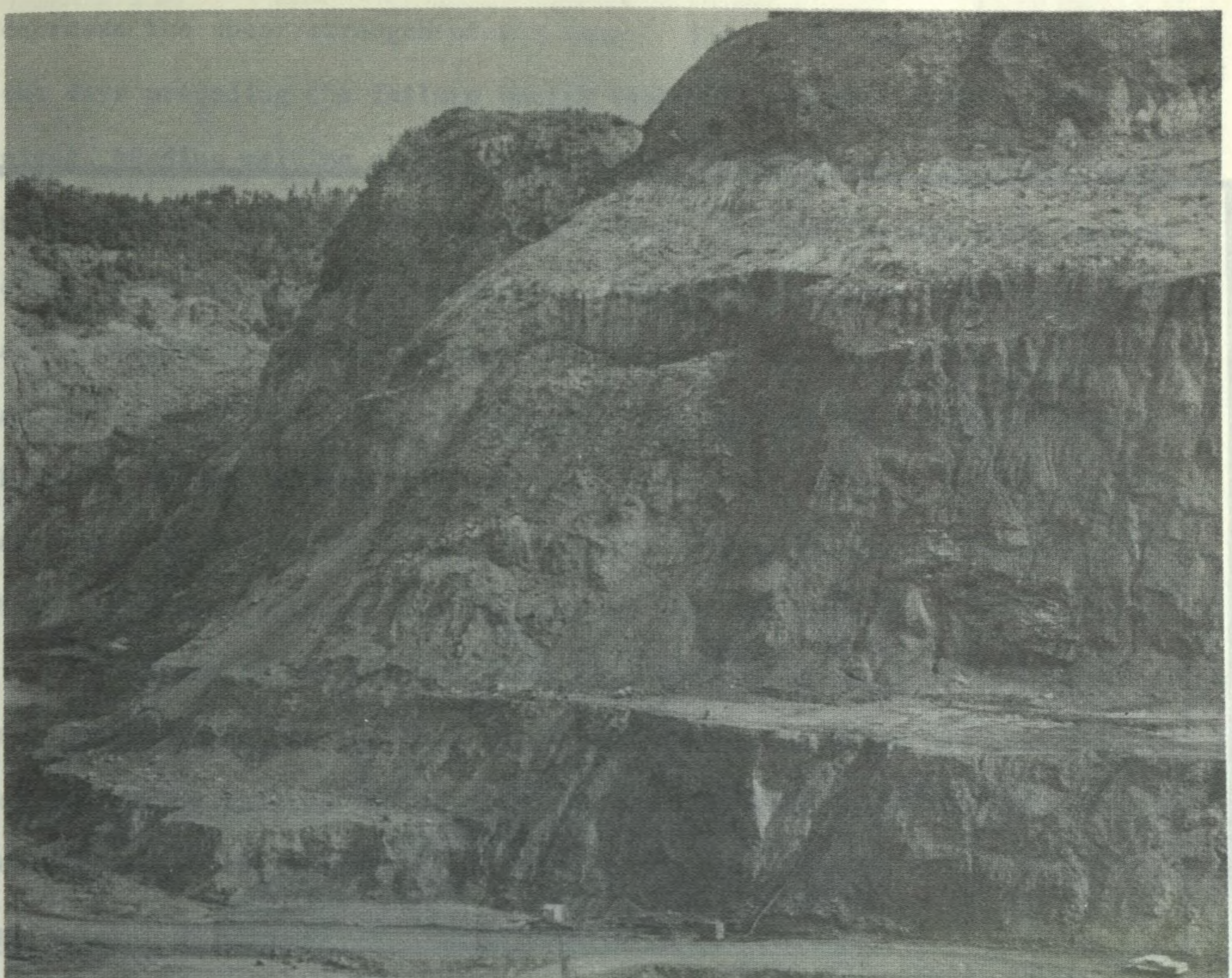


FIGURE 9 - ROTATIONAL SHEAR SLIDE IN A 215 ft HIGH SLOPE COMPOSED OF AN ALTERED ROCK COMPRISED MAINLY OF KAOLIN, QUARTZ & PYROLUSITE

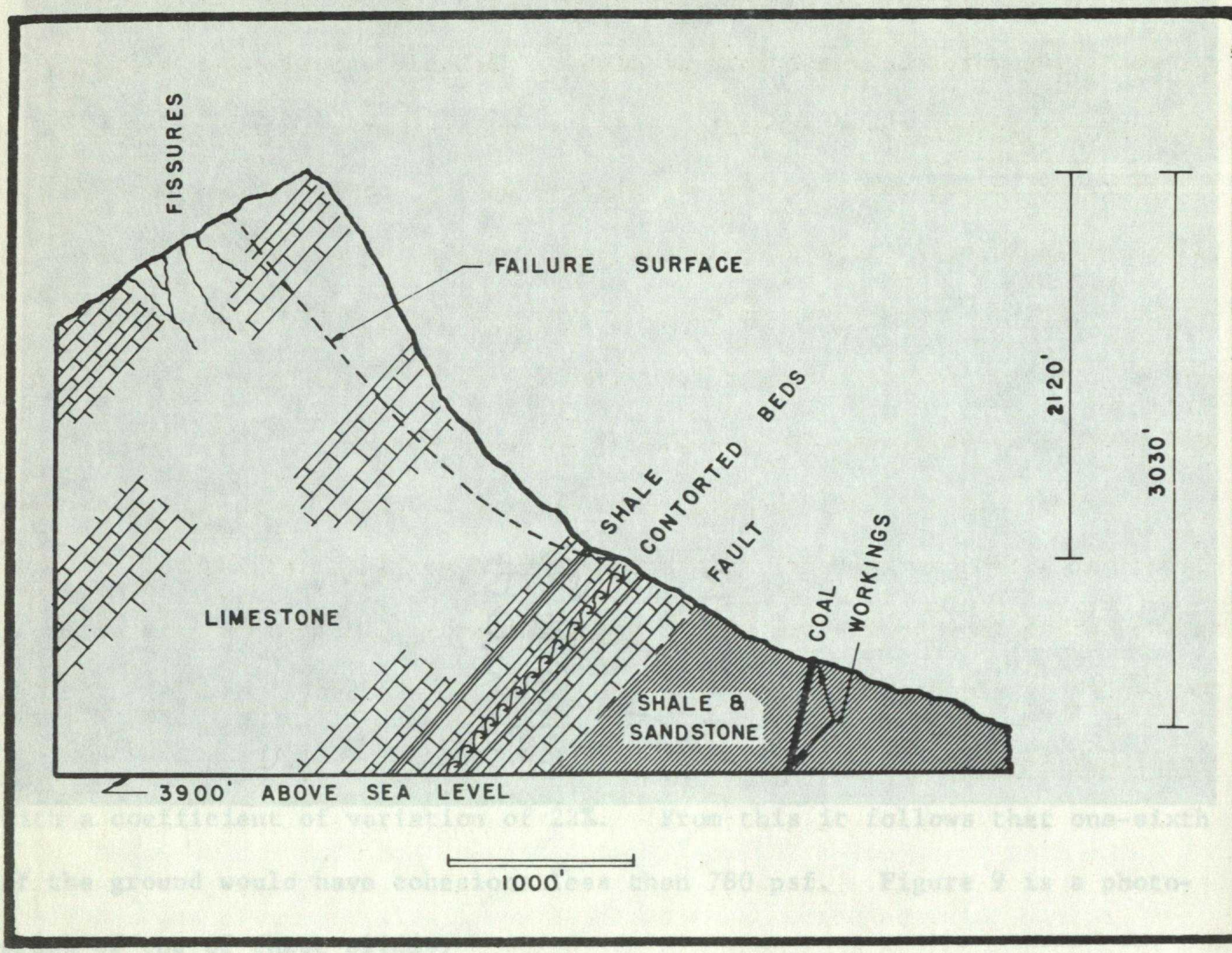


FIGURE 10 - A BLOCK FLOW FAILURE IN A SLOPE 2100 ft HIGH CONSISTING PRINCIPALLY OF LIMESTONE WITH SOME SHALE LAYERS

decrease the shear strength of the mass. First, the temperatures during the days preceding the failure varied from $70^{o}F$ during the day to $0^{o}F$ at night, causing melting of the surface snow during the day and freezing of the resultant water in the cracks at night. Second, ground water seepage added to the normal shear stresses. Third, compressible layers of shale and sandstone below the limestone at the toe of the slope probably were a source of additional slope deformation thus aggravating the stress distribution. Forth, underground workings in the coal at the toe added to this effect. Fifth, earthquakes that occurred during the preceding months could have marginally affected the strength of the rock mass.

Failure in the toe zone must have started at least seven months before the actual slide, as the coal in the underground workings had been self caving during this period of time. The slide zone coincided exactly with the length of the workings along the toe of the mountain.

Rotational Shear B

Fifteen slides were analysed in Proterozoic altered slate composed chiefly of an intimate mixture of very fine-grained kaolin, quartz, sericite and iron oxides and located usually in a synclinal structure. The rock could be classified as very weak, yielding, massive to broken. However, as the natural void ratio is of the order of 1.0 , it is again questionable whether this material should be classified as a rock.

Table 2 shows the description of the slides and the calculated cohesion. The calculated cohesion was based on an angle of internal friction of 34 degrees, which was determined in the laboratory, assuming the ground water level to be half the height of the slope and tension cracks to extend to a depth of one-quarter of the height of the slopes. The average cohesion obtained from these slides was 710 psf with a coefficient of

Table 2. - Altered Slate and Quartzite Slides

Slide No.	Height, Ft.	Slope Angle, Degrees	Cohesion, Psf
1	100	36	410
2	136	41	765
3	114	42	665
4	132	40	695
5	140	43	860
6	106	44	690

variation of 45%.

Figure 11 shows the cross-section through one slide zone as excavation proceeded over a four year period. In 1958 when the height of the slope was 85 ft a slide occurred that caused the crest to move down about 9 ft and out 4 ft. In 1961 with a slope height of 170 ft new cracks opened up in the crest of the wall, as shown in Figure 12. For a period of about two months the cracks opened at the rate of about 5 ins. per week with the side towards the excavation moving 1 in. per week downwards. There was no evidence of movement in the toe zone. In the spring of 1962 with a slope height of 195 ft a slide occurred with the maximum movement,measured close to the crest, being 16 ft horizontally outwards and 15 ft down; the movement of a hub located close to the toe of the slide was 12 ft horizontally and 3 ft down. Figure 13 shows the crest subsequent to this measurement.

Rotational Shear C.

Figure 14 shows the cross-section of a zone that produced several slides over a period of two years in the same material as for the Rotational Shear B slides. In 1962 the part of the slope between elevation 2130 and elevation 2060 produced a small slide. The appearance before the slide

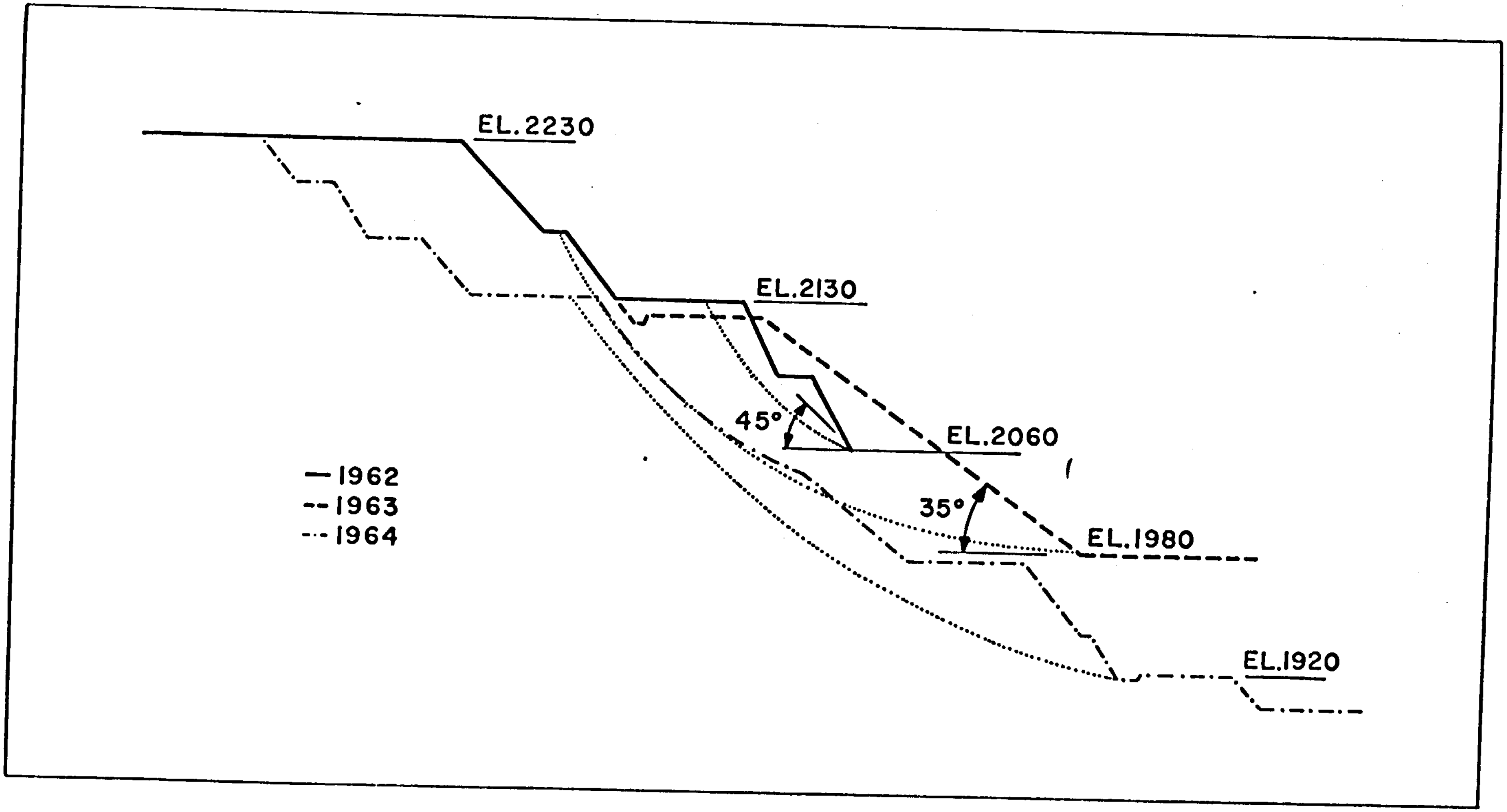


FIGURE 11 - CROSS-SECTION THROUGH A SLIDE ZONE IN A SLOPE WITH A HEIGHT THAT INCREASED FROM 170 ft TO 310 ft OVER A FOUR YEAR PERIOD IN AN ALTERED ROCK COMPOSED CHIEFLY OF KAOLIN, QUARTZ, SERICITE AND IRON OXIDES

FIGURE 12 - CRACKS IN THE CREST OF THE SLOPE SHOWN IN FIGURE 11 WHEN THE HEIGHT WAS 200 ft

FIGURE 13 - THE APPEARANCE OF THE CREST OF THE SLOPE SHOWN IN FIGURES 10 and 11 WHEN THE HEIGHT WAS 275 ft

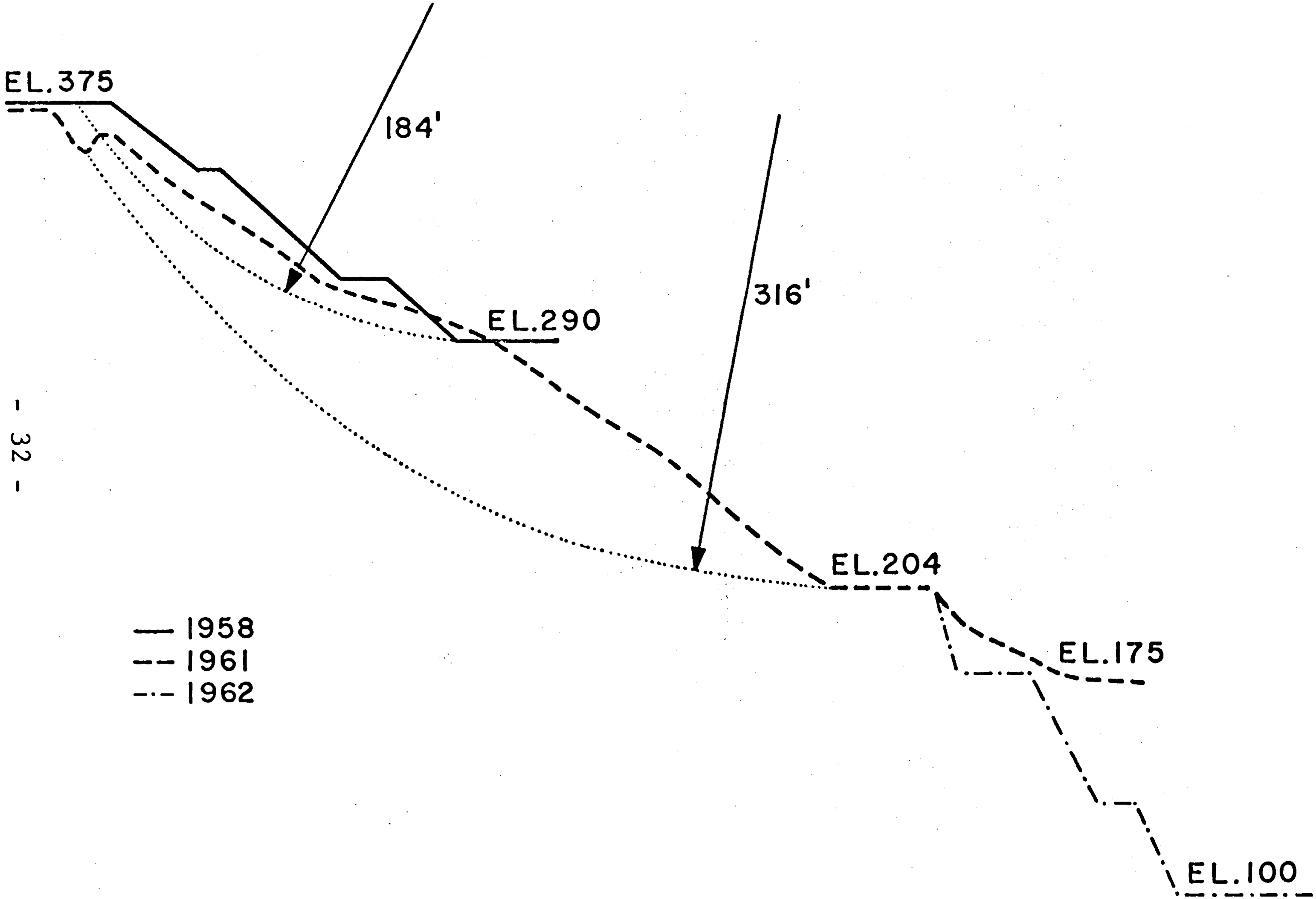


FIGURE 14 - CROSS-SECTION OF A SLOPE WHERE FAILURES OCCURRED AS THE SLOPE HEIGHT WAS INCREASED FROM 70 ft TO 310 ft

occurred of this section is shown in Figure 15 where a local drag fold can be seen. In contrast to the leisurely progress of the previous case, within a matter of 24 hours tension cracks appeared and the crest moved down some 5 ft.

In the spring of 1963 with a slope height of 140 ft sliding became quite general over a width of 400 ft at a slow and continuous rate. Figure 16 shows the appearance of one of the tension cracks in the crest by July of that year; however, the crest of the slide at this time had only moved down about 1 ft. The toe of the slide continued to move and by the end of September had moved about 50 ft horizontally.

During the summer of 1963 as part of the experimental remedial program, three horizontal drains were drilled through the slate into the underlying more porous quartzite with the hope of decreasing pore water pressures. A small flow was obtained which increased during rainy periods; however, by the end of July the continued movement of the slide broke the pipes and the drainage action ceased. These wells were tried in view of the ineffectiveness of vertical wells, 200 to 300 ft deep, in reducing ground water levels within a reasonable area of influence.

Excavation of the slide material during the winter, in spite of subzero weather, reactivated the entire slide zone as shown in Figure 17. By the spring of 1964 with the height of the slope 250 ft, a new larger slide started which moved quicker than the previous slides.

Rock Fall

There is an interesting, although undocumented, case of stabilization of a slope in a siltstone breccia. The slope was in an open pit mine in the south-west of the U.S.A. It was about 500 ft high at an average angle of 45 degrees. At this angle in spite of having been excavated in a series

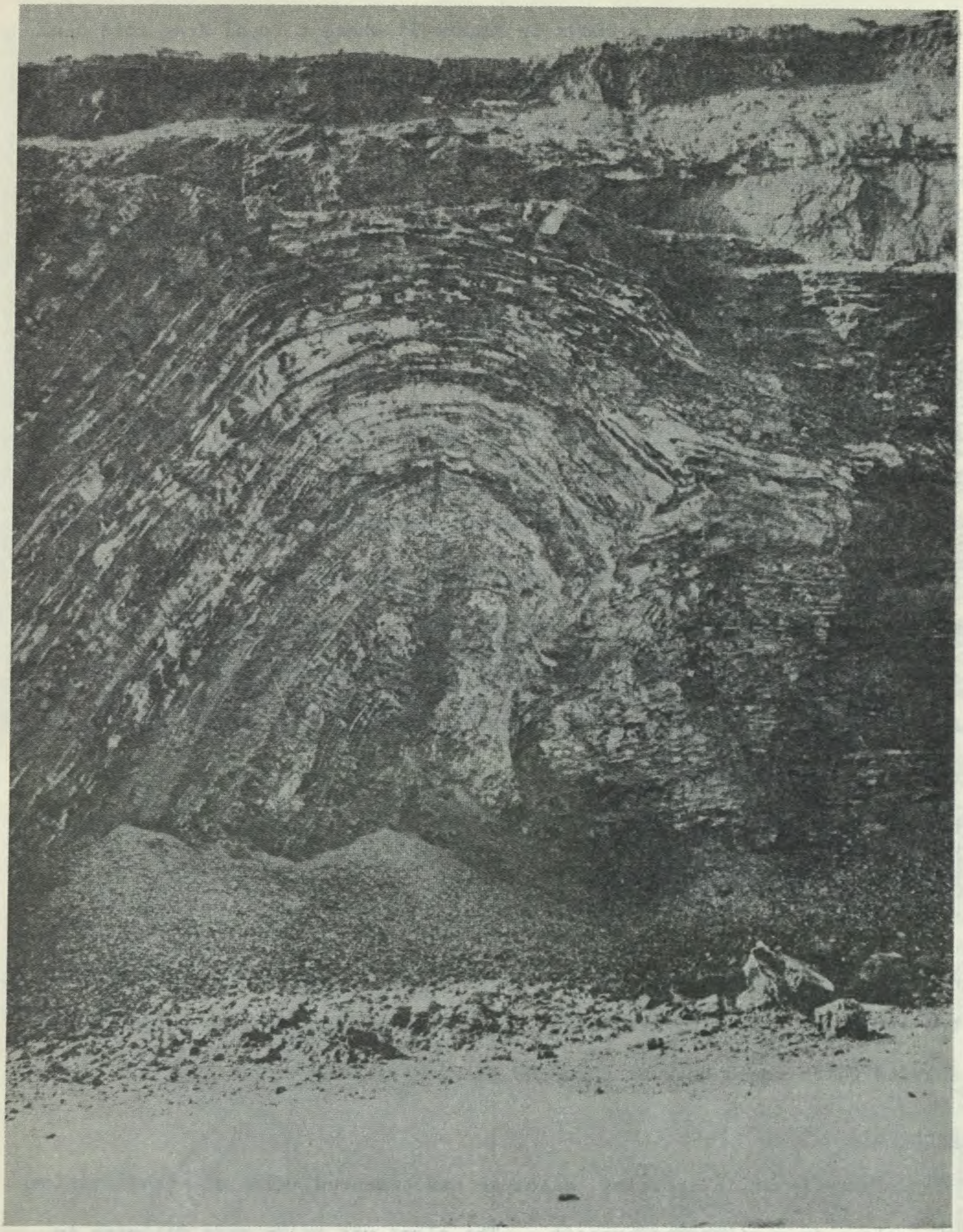


FIGURE 15 - THE APPEARANCE OF THE FACE OF THE SLOPE OF FIG.14
BEFORE FAILURE SHOWING A LOCAL DRAG FOLD

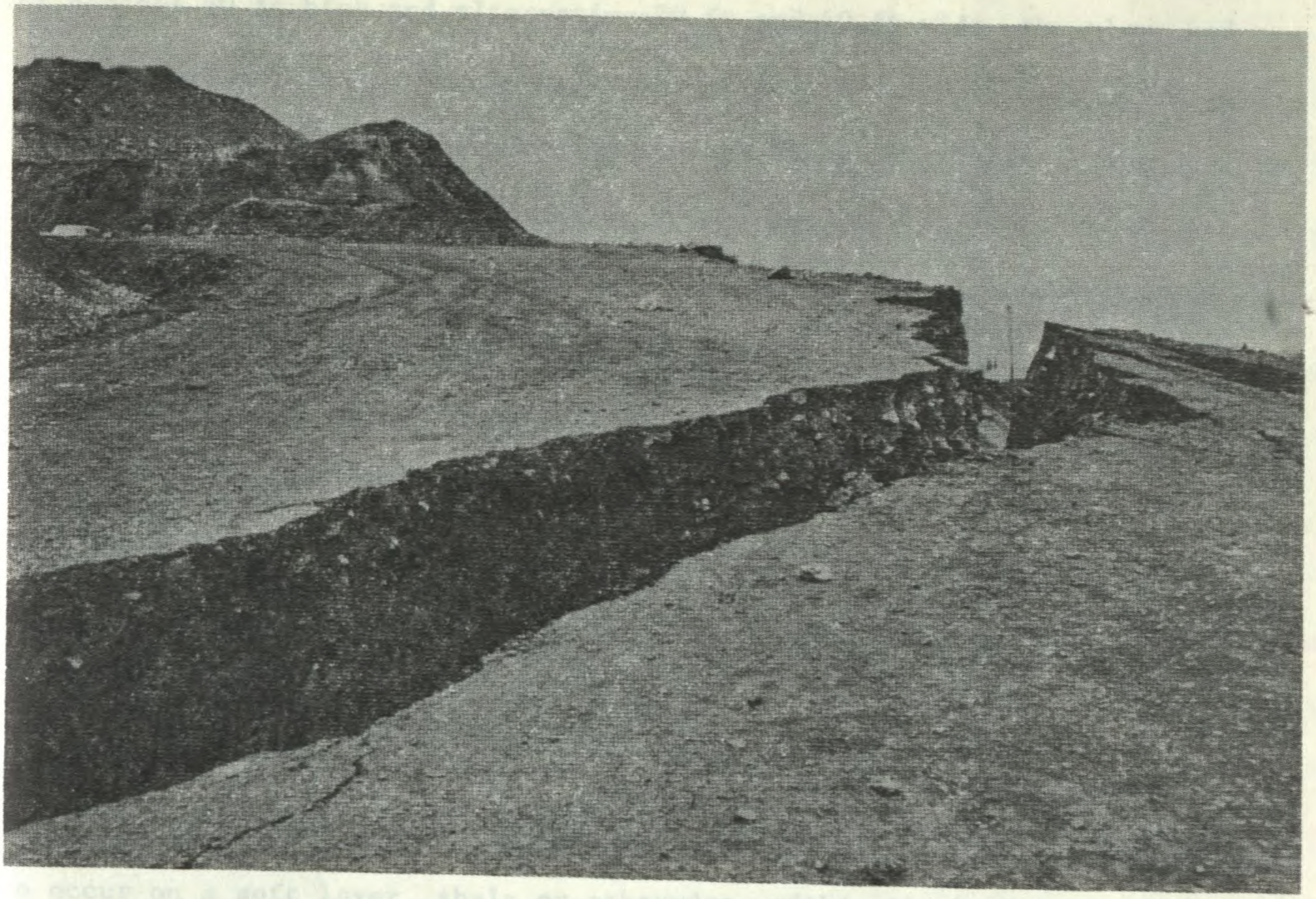


FIGURE 16 - ONE OF THE TENSION CRACKS IN THE CREST OF FIGURE 14 WHEN THE HEIGHT WAS 250 ft

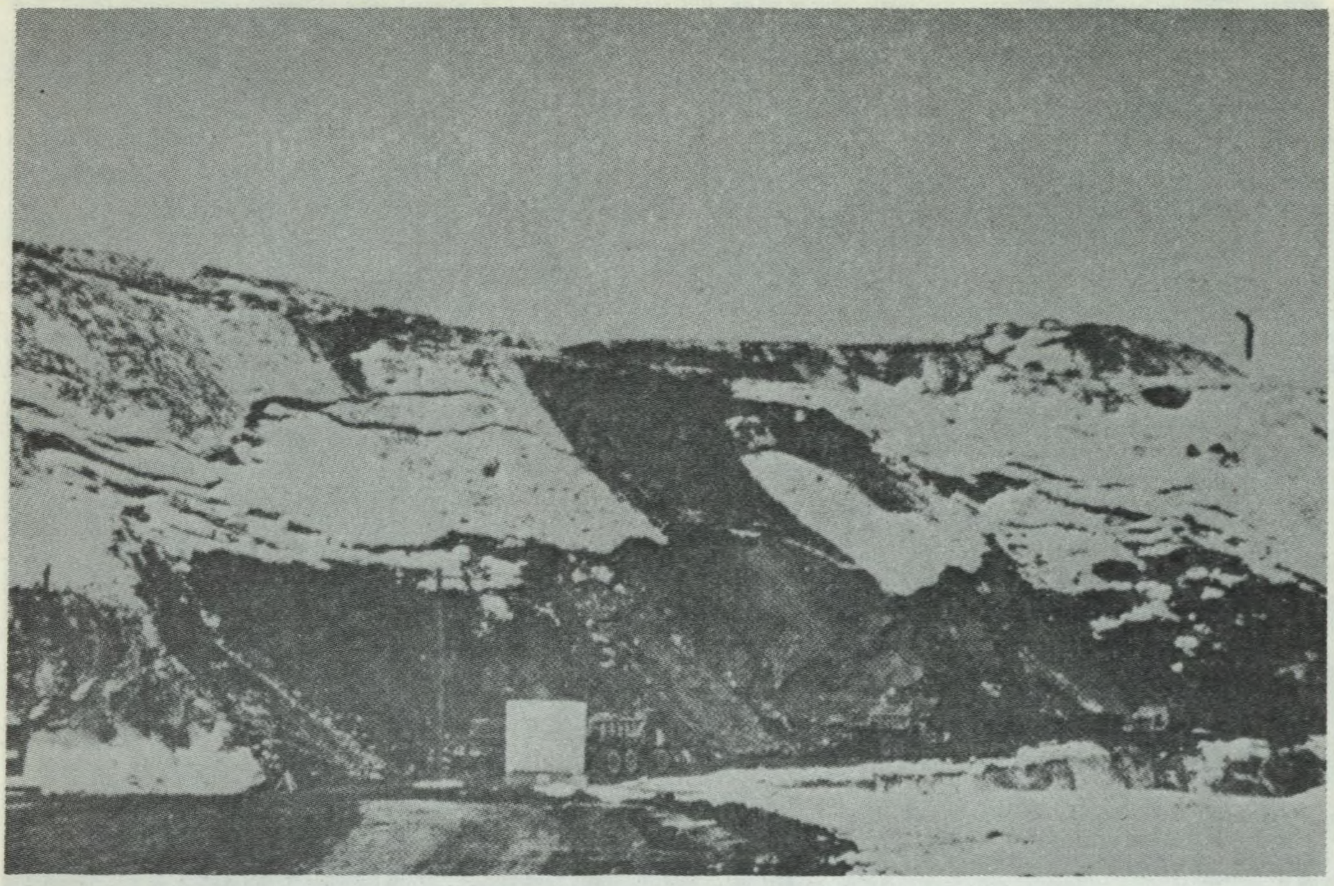


FIGURE 17 - NEW MOVEMENT, IN SPITE OF SUB-ZERO WEATHER, OF THE SLOPE SHOWN IN FIG.14 WHEN THE HEIGHT WAS 250 ft

of benches 40 ft high and alternating 10 ft and 40 ft wide, the slope had a continuous profile from toe to crest as a result of loose rock sluffing off the face and flooding the benches. Without these rock falls it is possible that the slope could have been cut to steeper angles.

A skipway had been cut into the slope. The resultant sluffing of the side abutments in the skipway cut caused considerable trouble. To remedy the situation rock bolts and mesh were tried without success; drilled-in concrete piles were tried also without success, and finally against their better judgment the abutments were grouted, which actually resulted in stabilization of the ground.

Plane Shear

Several case histories of plane shear failures have been already described in the literature (19,20). The common situation is for movement to occur on a soft layer, shale or otherwise, within the harder strata.

PLANNING OF ROCK SLOPES

In spite of the difficulties that lie in the way of explicitly designing slopes in shales, or indeed in any rock, there are many factors that can be taken into account which would improve planning and living with these slopes.

For example, our assumption that the analysis of stress is a key to predicting rock behaviour is, it should be realised, at this stage merely inherited from other related subjects but has not yet been proven to be the best procedure. The rationale of theory is to be able to generalize experience, e.g., if rock failure occurs and can be satisfactorily explained by the analysis of the stresses then this experience provides a good basis for using the same analysis to predict other cases of rock failure. If by analysing stresses this cannot be done, it serves no useful purpose.

On the other hand, some other factor such as deformation might be more easily related to failure of rock masses. It is for this reason that besides our field studies on in situ stresses, we are also measuring at several different sites rock deformation resulting from adjacent excavations and from the passage of time. For these measurements a variety of devices are used; however, the one that is probably most useful for slope studies and for monitoring purposes is the borehole extensometer.

Another aspect which must receive increased attention is the dispersion of strength properties that occurs in any structural material which must be assumed to be even more conspicuous in geological materials. It was pointed out above that where slope failures had been studied in the same formation the coefficient of variation of the strengths was as low as 22% in one case (which might be compared with job-mix concrete of 18% to 25% (21)) to as much as 45% in another case.

Figure 18 shows one way in which the dispersion of strength might be explicitly recognized in slope design. Curve I in this figure shows the required relationship between slope height and slope angle for substantially no slides (i.e. 1% probability of failure). Curve II shows the permissible relationship if a 10% probability of failure is acceptable, i.e. this could be interpreted as meaning that about 10% of the wall length would be involved in slides. Curve III illustrates another point: for the same probability of failure as Curve II the drawing down of the ground water level from 50% of the height of the wall to the toe elevation would make possible much higher slope angles for any given height of slope.

It is well known from theoretical analyses, as well as from experience, that seepage stresses are very important in slope stability. However, it is a mistake to assume that ground water levels can be controlled or even

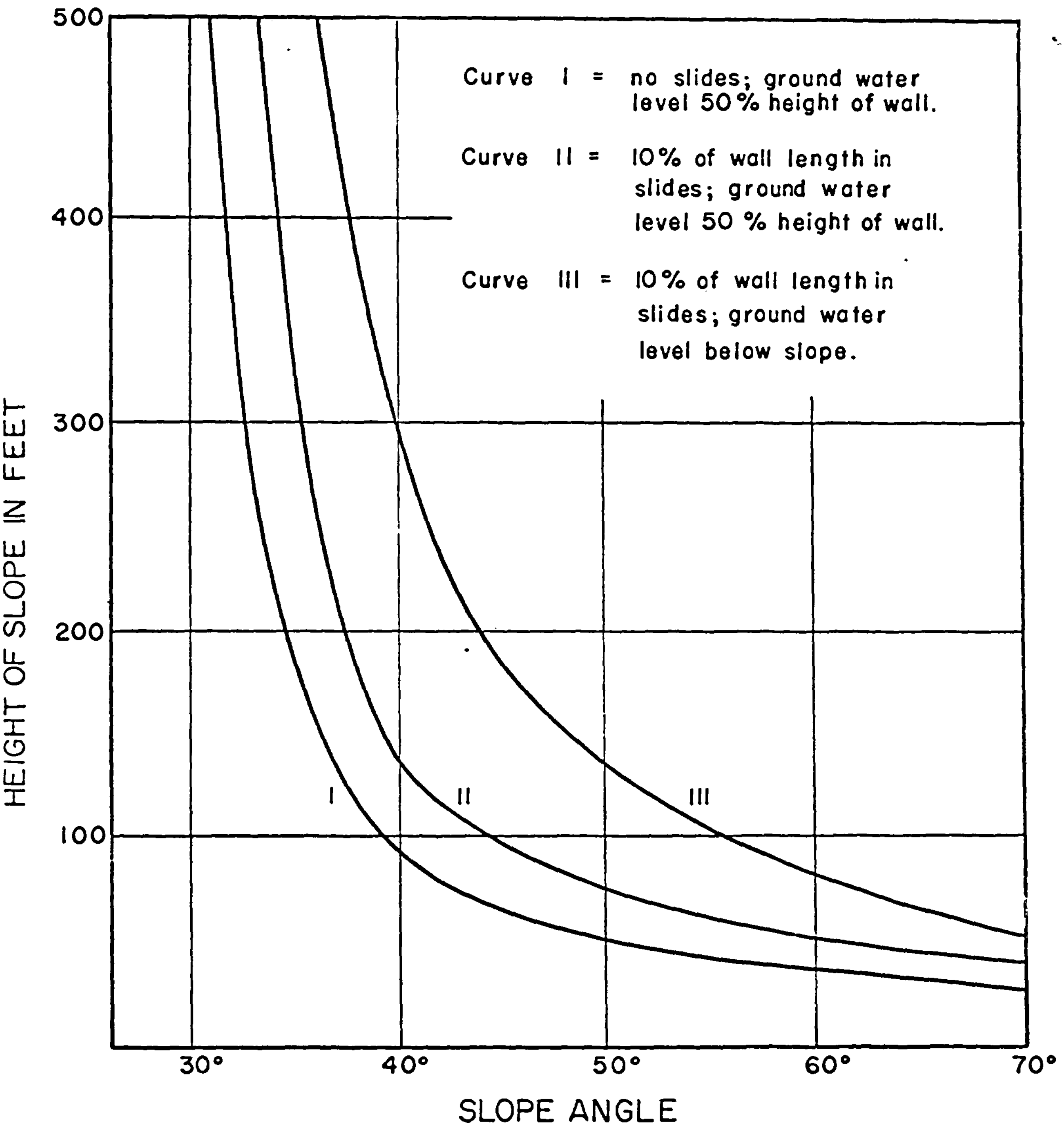


FIGURE 18 - TYPICAL STABILITY CURVES FOR YIELDING ROCK

measured with the same facility that has been experienced in soil work. As indicated above, it is possible in some rock formations to draw the water down 200 ft below the ground surface in a deep well and within 50 ft to have a piezometer indicating a water level 15 ft below the ground surface. Also, from experience we know that the large expense of a drainage drift can turn out to be a complete waste of money.

Possibly one of the most practical factors to consider in planning slopes is the use of controlled blasting. Construction people have known for a long time that if an excavation is to be made to fine tolerances which must be maintained, then close drilling is required. Although the benefits to be gained from using similar techniques for cutting permanent slopes has always been recognized, the additional expense seldom seemed to be acceptable. The analysis of the feasibility of this type of expense now seems to be producing a different answer for many organizations, both construction and mining, who are trying pre-splitting and perimeter blasting techniques (22).

For the special problems of shales, such as disintegration on exposure and swelling on decrease of load, special solutions must be provided for each case. Special coatings such as gunite, bituminous mixtures or possibly various types of new coatings, might be considered. Rock bolting and mesh have proven useful on some occasions.

CONCLUSIONS

1. A method of stress analysis for slopes in non-yielding rocks is needed. No solution for the stress distribution exists in the theory of elasticity. A computer solution using finite elements can be used until a simpler mathematical method is found.

2. Much research is required on the strength testing of rock masses and on the determination of the appropriate strength theory before an

explicit design procedure can be evolved.

3. The mechanical properties of shales extend over the full spectrum applicable to all rocks. Consequently, the types of slope failure to be expected in shales are essentially the same as those that can occur in other rocks. Some shales have special properties which essentially affect their strength; however, such peculiarities occur in other rocks as well. In addition, the dispersion of strengths in any one formation should be recognized, as is being done increasingly in concrete work, since it is an important factor for both safety and economy.

4. Case histories of slope failures show that some slides have occurred quickly whereas others have extended over a period of years. This is of interest as the time required for the failure process in some cases can be as important as the occurrence of failure itself.

5. Monitoring by measuring internal deformation should provide valuable information for research on the failure mechanisms and also for confirming the original appraised stability of planned slopes.

6. Controlled blasting probably is one immediate practical step that can be taken to increase stability of excavated slopes.

ACKNOWLEDGEMENTS

Data for rock classification have been provided by R.C. Parsons. Samples of Bearpaw shale were provided by R. Peterson, Prairie Farm Rehabilitation Administration. M. Gyenge and M.S. Ellis helped in the preparation of the manuscript.

APPENDIX. - READING REFERENCES

(1) Fillunger, P., "Drei Wichtige Ebene Spannungszustande des Keilformigen Korpers,", Z. Math. Physik., Vol. 60, 1912, pp.275-285.

(2) Terzaghi, K., "Theoretical Soil Mechanics," Wiley, 1943.

(3) Gyenge, M. and Coates, D., "Stress Distribution in Slopes Using Photo-elasticity - I," unpublished report, 1964.

(4) Coates, D., "Some Cases of Residual Stress Effects in Engineering Work," Proc. GSA Conference on State of Stress in the Earth's Crust,Elsevier,1964.

(5) Coates, D. and Freedman, M., discussions of "Petrofabric Techniques for the Determination of Principal Stress Directions in Rocks," by M.Freedman, Proc. Internat. Conf. on State of Stress in the Earth's Crust, Elsevier,N.Y., 1964.

(6) Larocque, G., "A Sonic Unit for the Determination of In Situ Dynamic Properties and Fracture Zones", Proc. 6th Symp. on Rock Mechanics, University of Missouri, 1964.

(7) Stears, J., "Evaluation of a Penetrometer for Estimating Roof-Bolt Anchorage," USBM RI 6646, 1965.

(8) Coates, D., and Gyenge, M., "Plate Load Testing on Rock for Deformation and Strength Properties," unpublished report, presented to ASTM Rock Mechanics Symposium, Seattle, 1965.

(9) Panek, L., "Determination of the Modulus of Rigidity of Rock by Expanding a Cylindrical Pressure Cell in a Drillhole," Proc. 6th Symposium on Rock Mechanics, University of Missouri, 1964.

(10) McClintock, F. and Walsh, J., "Friction on Griffith Cracks in Rocks under Pressure," US National Congress Applied Mechanics,Berkeley, 1962.

(11) "Glossary of Geology and Related Sciences," AGI, Washington, 1960.

(12) Coates, D. and Brown, A., "Stability of Rock Slopes at Mines," Bull. CIMM, July 1961.

(13) Rocha, M., "Mechanical Behaviour of Rock Foundations in Concrete Dams," Mem. No. 244, National Laboratory of Civil Engineering, Portugal, 1965.

(14) Serifim, J., "Rock Mechanics Considerations in the Design of Concrete Dams," - as #5

(15) Coates, D., "Classification of Rocks for Rock Mechanics," International Journal Rock Mechanics Mining Science, Vol.1, 1964, p. 421.

(16) Ladanyi, B., "Uniaxial Testing of Rocks for Classification Purposes," unpublished report, 1964.

(17) Peterson, R., "Rebound in the Bearpaw Shale, Western Canada," Bull. GSA, Vol. 69, 1958, pp 1113-1124.

(18) McConnell, I. and Brock, R., "Report on the Great Landslide at Frank, Alberta," Geological Survey of Canada, 1904.

(19) Schroter, G. and Maurseth, R., "Hillside Stability - The Modern Approach," Civil Engineer, June 1960.

(20) Lawrence, R., "Stabilization of Some Rock Slides in Grainger County, Texas," Economic Geology, Vol.46, 1951.

(21) Abdun-nur, E., "How Good is Good Enough," J. ACI, Vol.59, 1962, p. 31.

(22) Siebert, H. and Raitt, G., "Development of Rock Slopes in Metamorphic Rocks by Controlled Slope Holes," Proc. 7th Symp. on Rock Mechanics, Penn.State University, 1965.