STABILITY OF HIGH ROAD BANK SLOPES IN ROCK--
SOME DESIGN CONCEPTS AND TOOLS

by

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ABSTRACT

Described and discussed are the instruments and data analysis procedures used by the Bureau of Mines in the early stages of a continuing study of the manner in which an engineered predication can be made of the steepness of the angle and height at which a rock slope will safely stand.

The described instruments and procedures represent the beginning of a scientific effort to quantify experimentally the engineering and general significance of the attitude of rock joints, bedding planes and faults; the character of gravity and tectonic induced stresses in rock slopes with and without berms; slope geometry; ground water; and over-blasting as they affect the stability of a rock slope. Also discussed is the use of the pre-splitting technique of blasting to form a smooth and tight slope face relatively free of rock falls.

The reporting of these mining research studies may provide highway engineers with information regarding some new rock slope design tools which they may be able to use. However, this segment of rock excavation science is in its infancy, and much costly and time consuming work must yet be done before the various rock slope stability factors can be assigned numbers. Ultimately, technically sound and rational rock slope design criteria and procedures will be developed.

INTRODUCTION

Deep cuts in rock requiring the excavation of several hundred thousand or more tons of rock are becoming more commonplace as networks of wide multilane and limited access interstate highways spread across the nation. For these deep cuts the highway engineer is faced with the

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problem of estimating the slope angles for the sides of the cuts. If the selected angle is too steep, the uninterrupted usage of the highway and public safety is endangered by excessive and persistent rock falls or road obliterating slope failures and rock slides. If the planned slope is too flat, the projected cost of excavation may lead the engineer to resort to the unnecessary expense of choosing a longer route or driving a tunnel.

The mining engineer is confronted with a similar problem in open-pit mines. In these mines the steepness of the pit slope is a major economic consideration because the angle of the pit slope predetermines the quantities and the cost of removing waste material needed to safely recover ore.

Minimizing the amount of rock to be excavated is an objective common to both the mining and highway engineer in designing rock slopes. Solution of the problem requires that a quantified knowledge of the material properties, stress distribution, and effects of environmental factors be available. Unfortunately, the technology of quantifying the en masse properties of rock materials, the assessment of the effects of environmental factors, and the stresses to which rock slopes are subjected in open-pit mines and road cuts are, at this time, inadequately developed to positively establish the slope design criteria needed. Consequently, rock slope design practices have been and still are largely based on experience and "cut and try" methods, with slopes frequently made too steep or too flat. As such, large expenditures are often necessary to correct slope pitch after a failure occurs or for the unnecessary removal of more rock than required.

Recognizing the importance for developing a better engineering understanding of the factors governing rock slope stability, the Bureau of Mines and Kennecott Copper Corporation, independently, about 7 years ago, initiated investigations in the application of the principles of soil and rock mechanics in the design and control of rock slopes. In these initial studies, both groups assumed that the principles of soil mechanics, with possibly some slight modification, might be applied. However, the studies indicated that the application of soil mechanics was limited, in open-pit mines, to slopes composed of soil or to those parts of the active slide areas in which the broken and moving rock mass possesses physical properties similar to soil. Perhaps most important, was the indication that combinations of rock structures were primary factors in the stability of a rock slope with water seepage into and through the structures acting as an agent in promoting failures.

In 1960 the Kennecott Copper Corporation planned and started an extensive applied study of the stability of pit slopes using the Kimbley pit near Ely, Nevada, as a full scale experimental model. Because the program paralleled a general pit slope research study of the Bureau of Mines, Kennecott invited the Bureau to participate in the applied work as a joint Kennecott-Bureau effort. Some of the results of work completed and in progress have been published (5, 6, 13, 14, 22)[4/] or are being prepared for publication.

The following sections of this paper describes and discusses some of the open-pit mine rock slope design concepts and study procedures derived in the joint Kennecott-Bureau and related Bureau rock slope stability studies with the emphasis on those ideas that highway engineers may find useful in the estimation of the steepest slope at which road cuts may be expected to stand.

PRINCIPAL ROCK SLOPE DESIGN FACTORS

As we must consider a pit or road cut rock slope as an engineered structure, its design is primarily a problem in applied mechanics in which the factors to be considered, whether they be expressed by numbers established by measurements or estimations based on empirical knowledges, may be generally categorized as follows:

1. Stress geometry
2. Rock structure
3. Environmental conditions
4. Excavational procedures in sculpting the slope.

Stress Geometry

In theory and in practice the structural stability of any excavated opening in rock is partially dependent on the stress field, that is, the (1) state of the stress in the rock before excavation, (2) the stress distribution in the rock created by the excavation, and (3) the in-situ structural strength of the rock.

4/ Underlined numbers in parentheses refer to items in the list of references at the end of this report.

Important

All rock is in a state of stress before openings are excavated. The stresses may range from near zero to many hundred pounds per square inch, depending on the depth and configuration of the opening and on the magnitude of tectonic forces to which the rocks may be subjected.

The opening created by the excavation of a pit or road cut affects the stresses in the rock near the pit or cut. The stress distribution created by the opening is commonly referred to as a stress concentration, which is defined as the ratio of the stress at a point to the stress located outside the zone influenced by the opening.

The problem of evaluating the slope stability effects of stress concentrations created in the walls of an open pit or road cut becomes quite complex when the opening is large and irregular in shape, the elastic properties of different rocks in the slope vary widely, and planar discontinuities occur in the rocks in or near the slope walls.

Because of these complexities there is no satisfactory mathematical or physical model that can be used to predict these stresses or stress concentrations or their quantitative effect on the stability of a rock slope. Therefore, the concepts of the distribution of stress and the manner in which stresses affect the stability of rock slopes presented in this paper are based on (1) the qualitative stress distribution and concentration information derived from a laboratory study of body-loaded, two-dimensional, photoelastic model slopes; (2) some computed stresses as derived from measurements made in stress-relieved boreholes drilled horizontally at several sites into the walls of an open pit; and (3) some simplified assumptions as applied to the consideration of stress in, and in the vicinity of, an excavated surface opening as a pit or road cut slope. Briefly stated the general stress assumptions are as follows:

1. Vertical stress in rock which increases with depth from the surface is generally due to the cumulative body weight of the rock. It is usually about equal to the overburden weight density times the subsurface depth.

2. Horizontal stresses are developed by lateral constraint of rock loaded by the overlying material, by tectonic forces, or by both.

3. Horizontal stress should produce stresses in the slope which act parallel to the horizontal plane and tangentially to the rock slope surface. These tangential stress concentrations in a slope face may

favorably or adversely affect the stability of the slope, depending on whether these stress concentrations are zero, tensile, or compressive.

4. In slopes, that in plan are concave, the horizontal tangential stresses tend to be compressive, figure 1. In slopes, that in plan are convex, the horizontal tangential stresses tend to be tensile, figure 2.

5. Rock is relatively weak in tension, and tensile stress concentrations in a slope would promote instability; horizontal tangential compressive stresses, within reason, should assist in slope stability by increasing friction and restraint along fractures.

6. Stress concentrations around a pit or in a concave road cut slope can be tensile even though the two-dimensional stresses in the horizontal plane are compressive. This can be demonstrated by assuming a circular pit in which the rock (in mass) reacts as an ideal elastic medium and the stress concentrations created by the pit are equivalent to those created in a circular opening in a semiinfinite thin plate in a biaxial stress field where a compressive stress S is five times greater than a compressive stress T. Under these hypothetical conditions, the tangential stress concentration at two points on the edge of the opening is tensile and equal to -2T (1, p. 80.). An analogous condition in rock slopes could occur should a horizontal tectonic force of sufficient magnitude be superimposed on one of the lateral and horizontal components of the vertical overburden load. A stylized and exaggerated illustration of this condition is shown in figure 3.

7. Where any relatively incompetent rock or weakness created by joints, faults, or other planar discontinuities in a rock slope is subjected to tensile stresses, failures would be more probable than if these same areas were in confinement created by compressive stresses.

These are the assumptions and the hypotheses upon which a qualitative evaluation can be made of the effects of stress that, in part, govern the stability of rock slope.

As previously stated, a preliminary estimate of the distribution and the points at which the gravity induced stresses tends to be concentrated in a slope was obtained through a study of photoelastic models. Figure 4 is a simplified drawing of a photoelastic model representing a vertical slice from a hypothetical vertical-walled pit in which the gravity-induced unit strains are shown as stippled bands. Each of the bands represents a zone in which the strain is of a specific order of magnitude with the order of magnitude indicated by the circled numbers. The convergence of bands at points indicates the areas in which stress is concentrated. Thus it can be seen that stress is concentrated in the toe of the slope and that the stress in the floor near the slope toe is a tensional stress. The line through the inflection points of the top 3 bands denotes the plane of the angle of shear developed in the model. It can be seen that this plane starts at the toe of the slope wall and trends upward at an angle of 35 degrees to the vertical. In figures 5 and 6, this line is also at an angle of 35 degrees, even though the slope faces are now at angles of 75 and 60 degrees, respectively. As seen in figure 7, no such line of shear develops in a 50-degree slope.

Other model experiments have also shown that the magnitude of the stresses increases as the height of the pit wall is increased, but the angle of this line of shear remains at 35 degrees. It should be pointed out that the 35-degree angle developed is the angle representative of the homogeneous, isotropic material from which these models were constructed. It has not yet been established what this shear angle might be in a rock slope.

However, on the basis of the model-derived information, it is evident that, in a particular type of rock, the magnitude of the shear stress would increase proportionally with the increase in slope height but the angle of shear would remain constant regardless of the height of the slope. The model studies also show that as the height of a pit wall is increased, the increase in the stress is greater than the increase which would result from steepening the angle of the slope.

Figures 1 and 2 show diagramatically how the gravitational loading of the rock in slope walls and the resultant lateral stresses are transformed into a tangential stress at and near the slope surface.

Where the slopes form a concave surface, the tangential stresses act as a "hoop stress" and the blocks of rock in the pit wall tend to be squeezed together, figure 1. This creates an arch-like effect, thereby increasing the stability of the slopes. The converse is true in the convex-shaped slope face, figure 2.

A series of measurements made by the Bureau of Mines in the wall of an open-pit mine revealed that tangential stress concentrations exist in enclosed, concave, pit configurations. The results of these measurements and calculations are summarized in Table 1.

TABLE 1. - Stresses determined from measurements near the edge of the Kimbley Pit

Site	Depth from slope (feet)	Maximum stress (psi)	Minimum stress (psi)	Direction[1/] (degrees)
1	12 to 21	113	103	96
1	69 to 71	131	51	176
2	12.5 to 21	150	90	178
3	66 to 90	520	415	2
3	100 to 105	496	300	4

1/ Angle between the right hand horizontal and the maximum stress.

Because the boreholes were horizontal and normal to the pit slope, these determinations provide estimates of the stress in a plane parallel to the slope. Excepting the measurements between 12 and 21 feet at site 1, the maximum of the two orthogonal stresses is almost horizontal. Further, because sites 1, 2, and 3 are at deeper elevations from the original ground level, there is evidence that the tangential stresses increase as the distance from the ground level increases. Of note is the fact that the field stress determinations indicate that the expected horizontal stress at site 3 should be about 300 to 350 psi. The horizontal stresses determined at site 3 are about 500 psi; therefore the stress concentration (stress at site 3 divided by the horizontal stress) is about 1.5.

The above measurements were made in holes drilled horizontally at three sites into the walls of an open pit to depths ranging from 21 to 105 feet in length using the borehole deformation gage and stress-relief overcoring method developed by the Bureau. The instruments and the measurement and stress computational procedures used have been described and explained in published articles and reports (2, 8, 9, 10, 21, 22). Soon to be published (11) is an alternate method that may be used to compute stresses from strain measurements made by the overcoring method.

Information derived from the photoelastic models also shows that no overall reduction in slope stress is achieved by berming a rock slope face. As shown in figure 8 the level of stress at the roadway toe of the slope is no greater than the stress would be had the slope face been continued to its full height in a straight plane unbroken by a berm.

The extra rock that must be removed to include one or more berms appears to be a needless expense as far as the stability of a slope is concerned. ~~Thus the only purpose of a berm is to catch fall rock.~~ Generally berms to catch fall rock can be largely eliminated by using the presplitting or smooth-wall blasting techniques to form the slope face. Numerous articles and reports covering the theory and application of presplitting have been published (12, 15, 17, 18, 20), and the forming of slope faces in rock by the method is rapidly being accepted as a standard procedure.

However, the adverse role that horizontal tectonic forces may play in the achievement of a smooth face by presplitting has not been as equally well publicized. Referring to the previously stated simplified stress concentration assumption number 6 we see that a nonuniform stress field may result where a tectonic force is superimposed on the normal lateral component of the vertical stress. The affect of such a nonuniform stress field on the establishment of a presplit fracture between adjacent boreholes is depicted diagrammatically in figure 9. The length of arrows labeled "S" and "T" denotes the relative magnitude of the stresses as applied to the block representing a segment of rock in which holes have been drilled and in which presplitting charges have been detonated. Fractures established by the blast parallel the "S" (the trajectory of the greater stress level) direction with fractures established between holes in the "S" direction and not between holes in the "T" direction. To establish fractures between holes in the "T" direction the holes must be loaded with much heavier charges than the holes in the "S" direction. Thus to be able to specify hole spacing and charges to successfully presplit at any particular site, the planning engineer must know whether the horizontal stresses at the site are uniform or nonuniform.

The Bureau has found relatively large horizontal stresses in igneous, metamorphic and sedimentary rock areas. In some instances these stresses are not uniformly distributed in all directions (8, 3).

The level and trajectory of the tectonic stresses in a particular area may be established by the borehole stress relief method developed by the Bureau. Generally the information can be obtained by stress relieving one hole drilled vertically into the rocks exposed at each suspected site to such depth as required to penetrate below the zone affected by sheeting or surface fracturing. Where rock is solid and unaffected by sheeting or surface fracturing, the hole need not be more than 3 or 4 feet deep.

Rock Structure

In the initial phases of the slope stability investigation conducted by the Bureau, a detailed study of slope failures in open-pit mines revealed that the incipient plane of failure and the plane along which movement of the mass of rock ultimately took place was almost always a planar or structural discontinuity in the rock. This observation led to a study of natural rock slopes; and in all the slopes studied the rock structural features were found to be the primary factor governing the stability of the rock slopes. This conclusion was also reached by Terzaghi (19) in his assessment of factors governing the stability of slopes in unweathered rocks. Therefore, the Bureau has taken as a basic working hypothesis the idea that rock structures, as joints, bedding planes, schistosity, faults, etc., are measureable features which can be utilized as a means of predicting the steepness of the angle at which a slope in rock will stand.

Thus the knowledge of the orientation and character of these planar rock structures is necessary in order to establish the steepest angle at which a rock slope will stand. Studies have shown that the critical height and steepness of slopes in relatively unweathered rock may be dependent more on the orientation of the planar defects than on the strength of the rock itself. The engineering concept, that all rocks are similar, probably stems from the idea that there is relatively little variation in the laboratory-determined compressive strength, and to some extent the shear strength, of most rocks encountered in rock excavational operations. This concept relies too heavily on strength factors determined by the laboratory testing of small samples. These laboratory results can differ greatly from the en masse characteristics that rocks exhibit in place. Hence, the laboratory-determined values of rock strengths must be used with some reservation in computing the strength and stability of slopes because of observed effects of joint planes and other planes of weakness in rocks.

Joints subdivide rock into individual blocks which almost fit each other, and the cohesive bond across an uncemented joint is assumed to be zero. Joints may be continuous or discontinuous and generally form a three-dimensional network that transforms the rock into a cohesionless aggregate of cuboid blocks somewhat comparable to closely fitted blocks in a dry masonry wall. The blocky nature of the rock in an excavated slope is shown in figure 10. Unlike a dry wall, the joints in rock are commonly filled with decomposition materials that are prone to turn into a grease-like substance when wet. Thus, any set, or intersection of two or more sets, of more or less continuous joint planes is a potential surface along which sliding can occur. Hence, the orientation of persistent sets of joints with respect to an actual slope must be seriously considered.

These same statements apply even more to fault planes, regardless of how great or small the displacements along them appear to be. Joint sets, bedding planes, and fault planes that dip into the slope face will generally have a minor effect on reducing the stability of rock slopes. Conversely, those sets of joints, bedding planes, faults, and other planar discontinuities with dips that are oriented toward the open face of a slope must be carefully considered. The detrimental effect on the stability of a slope increases as the strike of the joints becomes more nearly parallel to the slope wall. The fact that rock is almost always jointed in a preferential direction is all the more reason why it is more than probable that any slope will have one or more sets of joints adversely situated in relation to orientation of slope face.

Therefore, it is essential to collect as much information concerning the dip, strike, location, and character of all bedding, and joint and fault planes, especially the faults, as is physically possible. In addition, the location of breccias and decomposed, weak, permeable materials should be given special attention.

The major part of field data on the rock structures may be collected by conventional geologic mapping methods augmented by data obtained through a study of aerial photographs by the use of photogeologic techniques applied to the mapping of joints as reported by Hough (4). Also where suitable drilled holes are available additional joint information may be obtained with a borehole camera. The borehole camera and its use has been described by Rausch (13), and Hubbard and Rausch (5).

The dip and strike information may be represented in lower hemisphere equal area (polar) contoured plots as described by Robinson and Lee (16) or recorded on computer punch cards which can be machine sorted by strike and dip and histograms or contoured equal-area plots made as reported by Hubbard and Rausch (13).

These plotted or contoured diagrams then serve to show the engineer the spatial orientation of the planar discontinuities he must consider as potential sliding planes in designing a rock slope.

A good deal of the tedious work and errors in plotting the dip and strike point information on polar (lower hemisphere) equal area diagrams may be eliminated by adopting a system of field data recording and a plotting aid developed by the Bureau.

A compass having a full circle 360 degree azimuth is used instead of the quadrant type of circle division normally used by geologists. The dip point plotting device developed by the Bureau is shown in figure 11. Inscribed on the backboard is a 360 degree circle. In the center of the circle a pivot hole is provided into which the pivot pin on the T-shaped spaced piece fits. On the diametral or longest part of the T-shaped piece there is inscribed center to right and to the left from 0 to 90 the dip angle units corresponding to the polar circles on a Lambert's equal area polar net. (Note: To provide support for the pivot pin the central 0 to 5 degree is omitted on the scale as structures having a dip of 5 degrees or less can, except in very rare circumstances, be ignored as having little or no significance on the stability of a slope). The method that may be used to derive the spacing of the polar circles to fit any selected size plotting circle is shown in figure 12.

The method of field notation using the 360 degree azimuth compass is as follows: Strike is always read from the north end of the compass needle as the direction in which the compass is pointed and the operator is facing, and dip as being either to right or to the left of the direction the operator is facing along the strike plane. A comparison of the manner in which the strike and dip data is recorded in the field notebook using the quadrant and Bureau system is shown in Table 2.

TABLE 2. - Comparison of quadrant notebook strike and dip notation with 360 degree method used by Bureau of Mines

Bureau System		Quadrant System	
Strike	Dip	Strike	Dip
225	R65	S45°W	65°NW
110	R70	S70°E	70°SW
225	L65	S45°W	65°SE
110	L70	S70°E	70°NE

Using the Bureau plotting aid and notation system the office procedure in plotting the field data 225 R65, shown in Table 2, is as follows: Fasten piece of tracing paper to board; mark North or zero point; insert pivot pin on T-shaped piece into center pivot hole; insert pencil point in hole in the pointer and inscribe outer 90° dip circle; turn pointer to 225 degrees; plot dip point at 65 degree mark on the scale on right hand side of pointer. Additional points are plotted in same manner with dip points plotted either to right or left of the pointer in accordance to field book notation of R or L. See figure 13, showing the plotting of 225 R65 and 225 L65 and figure 14 showing the plotting of 110 R70 and 110 L70. Using this plotting aid and notation system a nonprofessional worker can be trained to accurately plot rock structural data on a Lambert's lower hemispherical equal area net in a few minutes.

Contouring the plotted information as shown by Robinson and Lee (p. 28, 16) is done by any one of the accepted methods used and known to geologists.

To read the contoured plot the tracing paper is placed on the plotting board with the zero point on the paper alined with the zero on the board and the T bar is then used to determine the strike orientation of the contoured dips.

Figure 15 is the joint system and bedding plane mapped in the "Hungry Horse" rock slope shown in figure 10. It diagrammatically illustrates the joint plane (J1) that governs the stable altitude of slope in this specific road cut.

Establishing the steepness of rock cut slopes on the basis of orientation of the planar structures relative to the slope face will permit the highway engineer to design stable slopes with a reasonable degree of confidence as an indeterminate safety factor is "built in" because cohesion along the planar structures is considered to be zero.

A more refined system of establishing rock slope angles will eventually be developed when through continuation of slope stability research engineers will succeed in developing the necessary mathematical or physical models by means of which the effects of stress distribution and other factors affecting slope stability in rocks may be quantitatively equated.

Environmental Conditions

In many cases, the toe of a slope is located below the water table, and the influence of hydrostatic pressure is not clearly understood nor properly estimated. However, slope failures due to rapid and wide fluctuations in the elevation and configuration of the water table are quite common.

In relatively unweathered, jointed, and fissured rocks, the volume of the open joints in which the water collects and along which the water travels through the rock is very small as compared with the volume of rock located between the joints. Hence, the rate at which the water moves through a rock mass is generally much slower than the rate of movement through pervious, unconsolidated sediments. Therefore the vertical fluctuations of the water table in a rock mass are much more pronounced than the variations of the level of a water table in pervious sediments. Thus, the vertical distance between the lowest and highest position of the water table in a rock slope may vary by several tens of feet or more as compared with the few feet of variation noted in a pervious sediment. The character of the sometimes quite rapid change of the water table in a rock slope is shown by comparing line A-F with line A-G on figure 16. In reality, the water table is not as well defined in rock as shown by the dry season line, A-F, and the wet season line, A-E, because the spacing of joints may vary from place to place and water may rise to different horizons in adjacent observation wells.

The following discussion points out some of the reasons why water in pit or roadcut slopes can present problems that are sometimes difficult to overcome.

During rainy seasons or spring snow melts, part of the water is temporarily retained in the weathered top layer, and the remainder flows as runoff sometimes toward and over the edge of the slope. Before it reaches the lip, the runoff crosses the upper surface (C-B, figure 16) of the wedge-shaped body A-B-C. Because of the shearing stresses commonly prevailing in this segment of the pit slope, the joints may be wider and more numerous than those in the rock located farther from the face of the slope. Hence, the quantity of water which can enter the joint system in this wedge, per unit area of its top surface, is much greater than the amount that enters the joint system elsewhere. Consequently, the water table in the wedge may rise temporarily to a position like that shown by the line A-G on figure 16. This added height of water thereby exerts, onto the walls of the pit, an added pressure equal to the hydraulic head created by the increase in the height of the water table in the wedge-shaped segment A-B-C. This hydraulic pressure corresponds to the pore-water pressure in soil mechanics and is hereafter called joint-water pressure. If the rock near the toe of the slope is already stressed to a near failure point, the added joint-water pressure may be sufficient to cause the slope to fail. Failure will start at or near the toe, and, as a result of the initial failure, the rock located above the seat of failure will be deprived of support and will fall because of its own weight.

Under certain climatic conditions, an even more destructive joint-water pressure can develop behind slopes. During spring thaws, the night temperature is usually great enough to freeze water. Ice forming in the joints on the face of the slopes effectively seals the joints, thus damming up the thaw water, as shown by line H-G in figure 16. This increased head of water, although generally only temporary, may cause the slope to fail.

The effects of water as a lubricant along gouge-filled fault planes should also be considered where the dip of the plane is toward the open face of a slope.

The effectiveness of surface and underground drainage in stopping or reducing slope movement has been demonstrated many times. In some cases, however, the expenditure of excessive amounts of manpower and money to obtain effective drainage may not be warranted.

Excavational Procedures in Sculpting the Slope

Among the procedures used in the excavation and operation of a slope, the practice of overbreaking at the toes to reduce shoveling difficulties should be considered as a possible cause of slope weakening. The unit strain lines shown on figures 5, 6, 7, and 8 as dotted bands show the area at the toe of a slope or bench to be more highly stressed than other contiguous parts of the slope. This highly stressed toe is the area where failure of a slope is most likely to start. The shattering of the rock in the toe of the slope by overbreaking unnecessarily weakens the slope at a point where a concentration of stress may have already reached a critical level. This shattering is an aspect of slope stability research that has received too little attention. However the practice of shaping slopes by the presplitting or smooth wall blasting technique can greatly reduce the weakening of the toe of the slope by vertical shot overbreaking.

CONCLUSION

Through continued studies and research, engineers will eventually develop the necessary mathematical or physical models that will permit them to quantify the effects of stress distribution and other factors affecting the steepness at which rock slopes will safely stand. In the meantime and until such models have been developed, the engineer must continue to design rock slopes and the method of basing the steepness of rock cut slopes on the orientation of planer structures, as described in this report, is a possible interim method that may be used to design rock slopes. However, the fallibility of the method has not, as yet, been fully established. Much time consuming work must yet be done before methods of measuring the effect of planar structures, stress, and other factors affecting the stability of rock slopes have been developed and proven.

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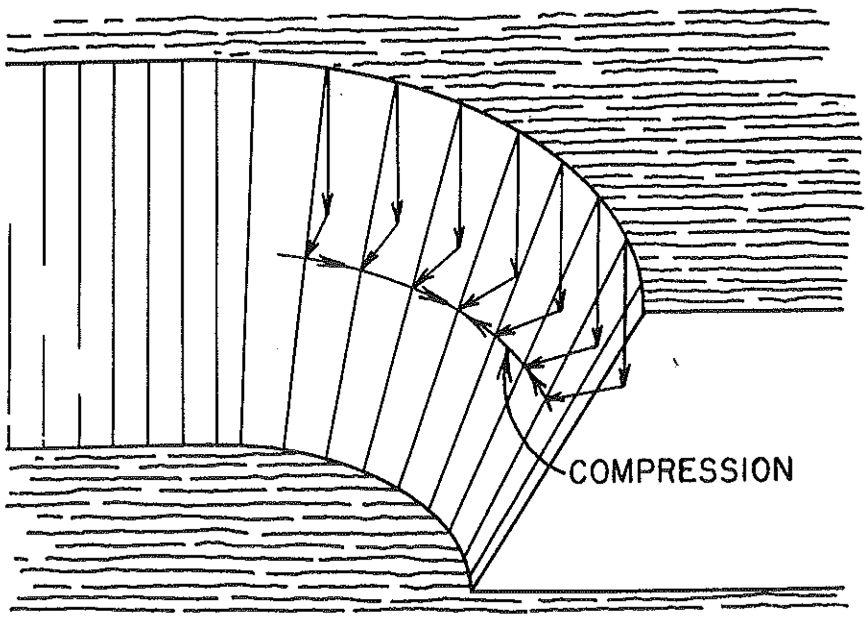


Figure 1. - Strengthening effect in a concave-shaped wall of a slope as the result of horizontal tangential stress created by the lateral resultant of the wall burden load.

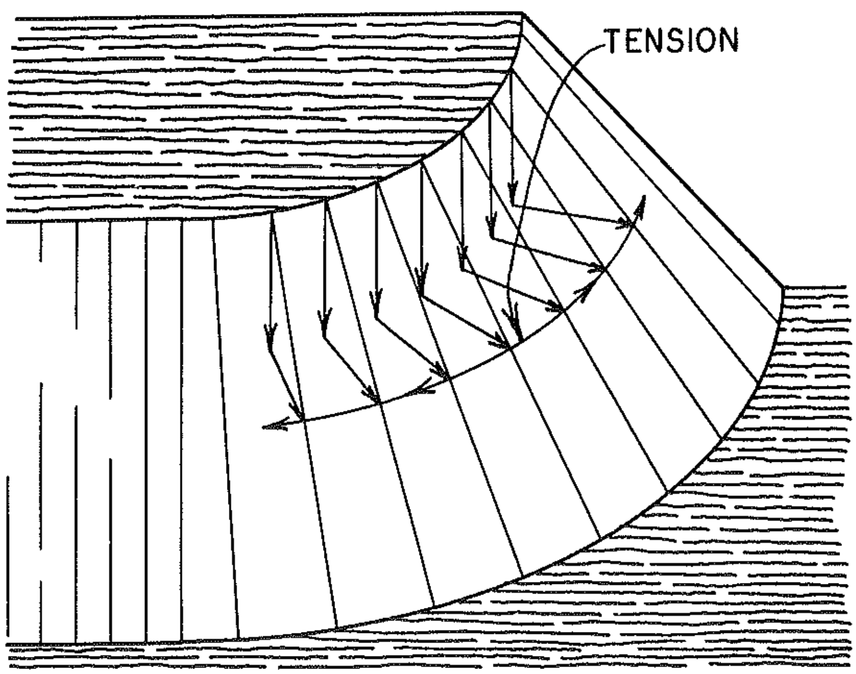


Figure 2. - Weakening effect in a convex-shaped wall of a slope as the result of the horizontal tensional stress created by the lateral resultant of the wall burden load.

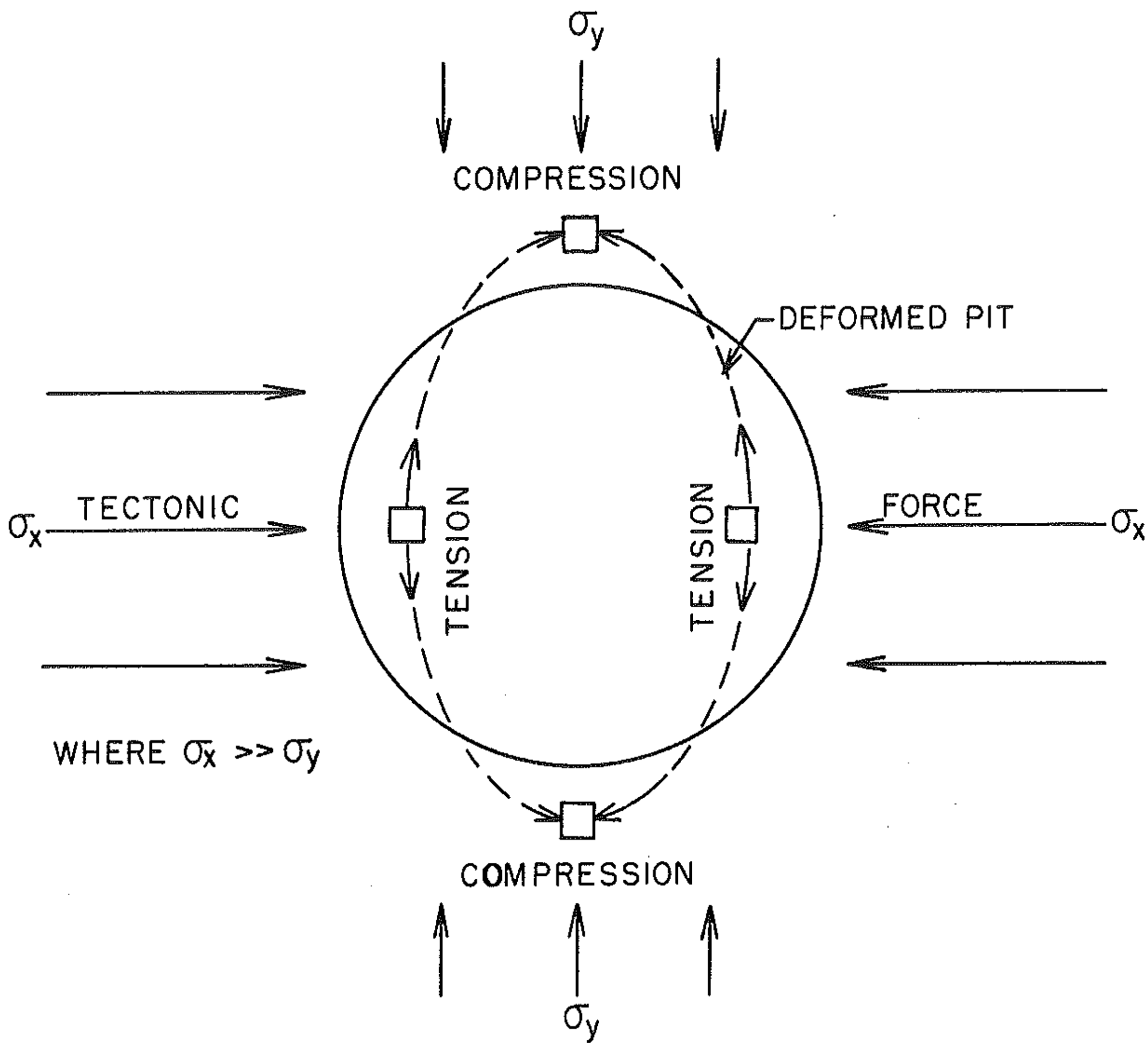


Figure 3. - Exaggerated depiction of deformation of a circular pit resulting from an excessively high horizontal tectonic force.

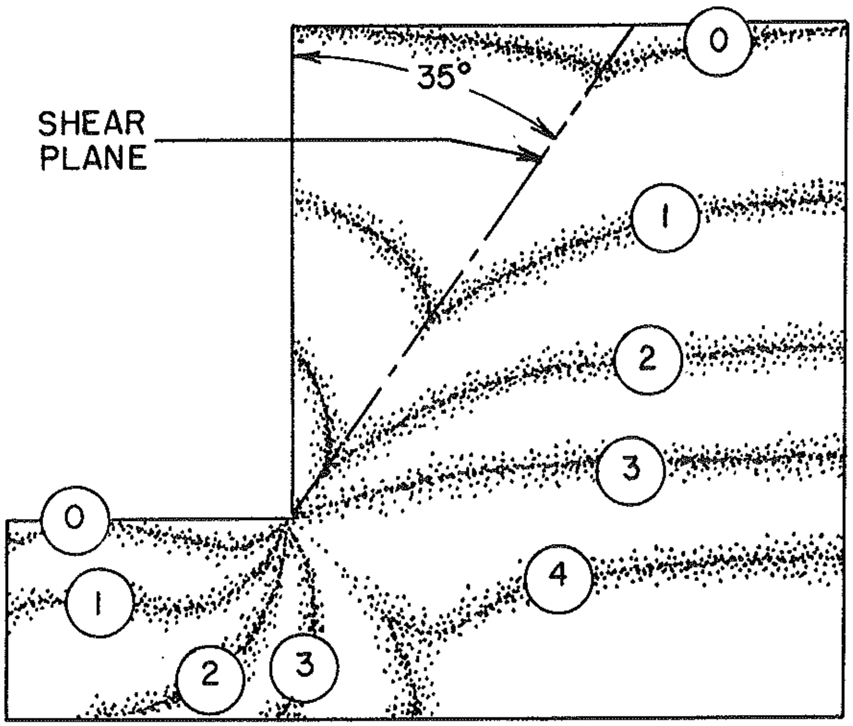


Figure 4. - Levels of strain and shear plane developed in a photoelastic model of a vertical slope.

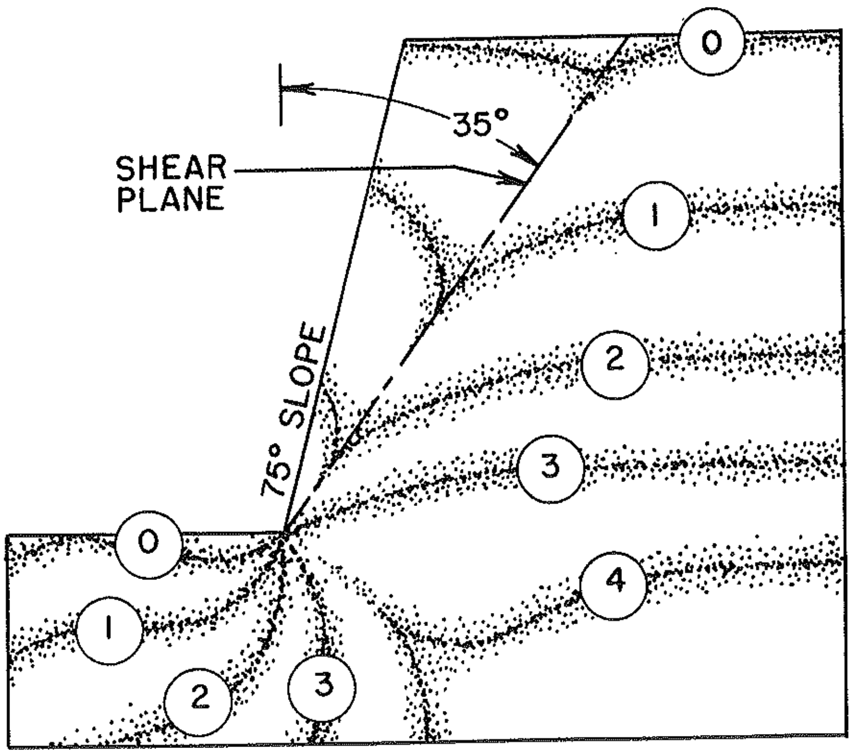


Figure 5. - Levels of strain and shear plane developed in a photoelastic model of a 75-degree slope.

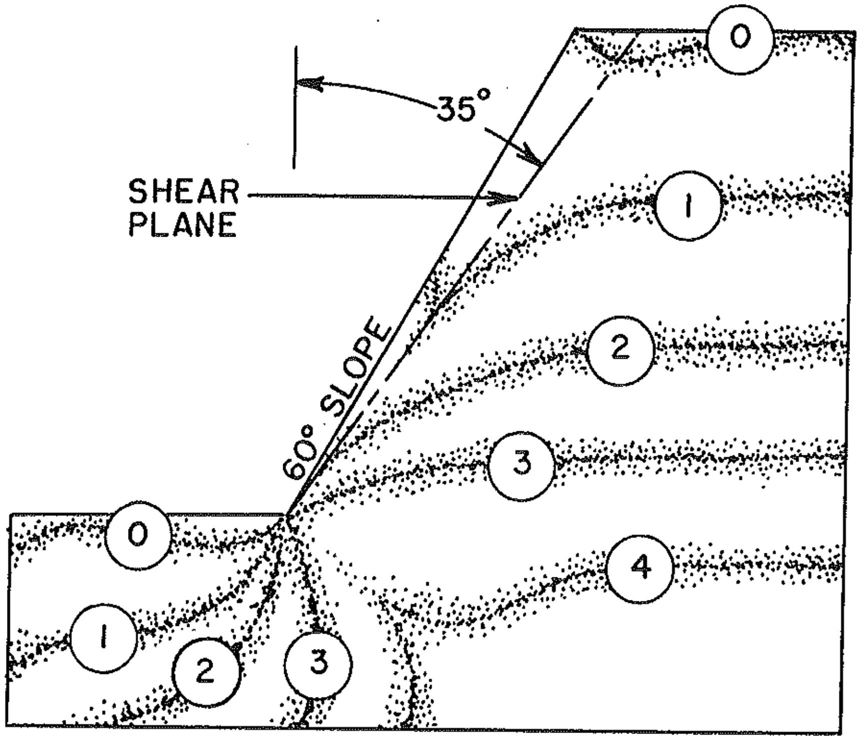


Figure 6. - Levels of strain and shear plane developed in a photoelastic model of a 60-degree slope.

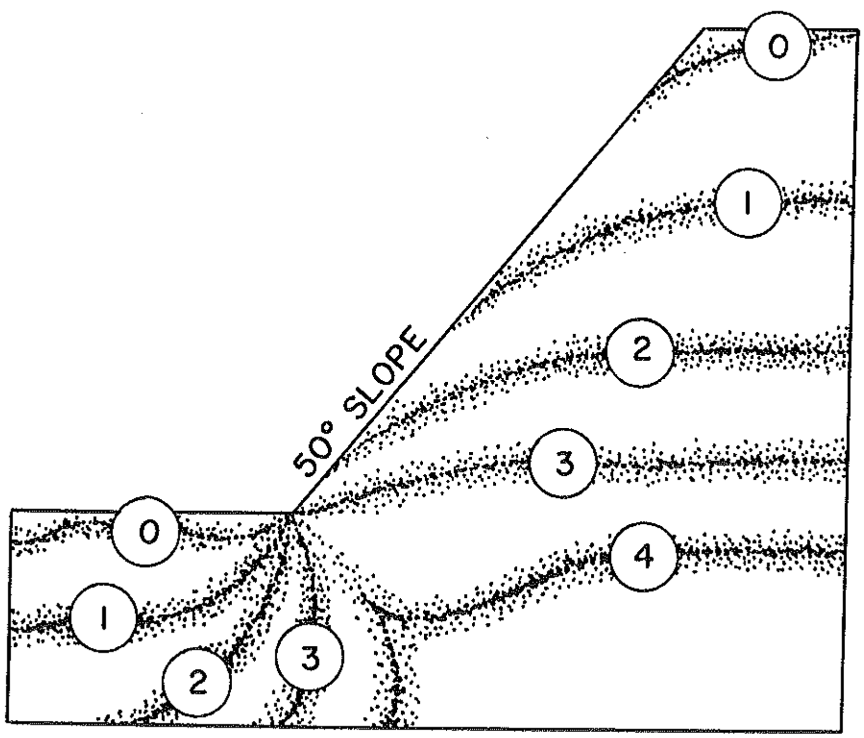


Figure 7. - Levels of strain developed in a photoelastic model of a 50-degree slope.

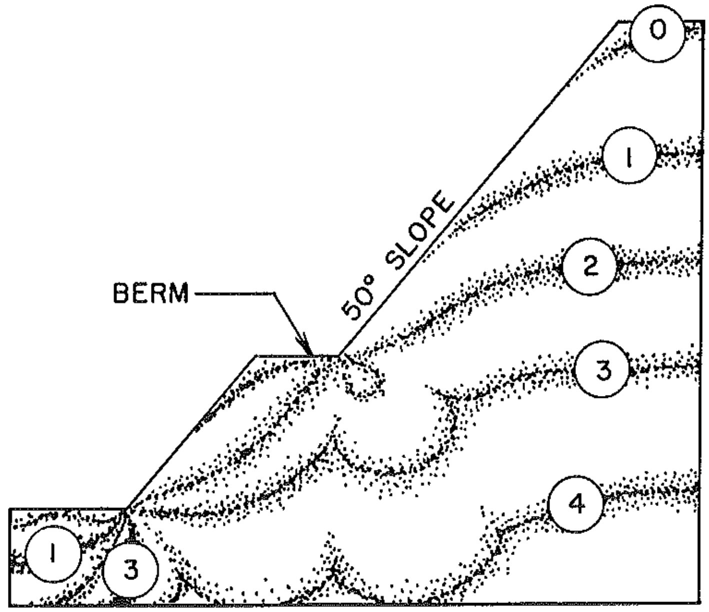


Figure 8. - Levels of strain developed in a photoelastic model of a bermed, 50-degree slope.

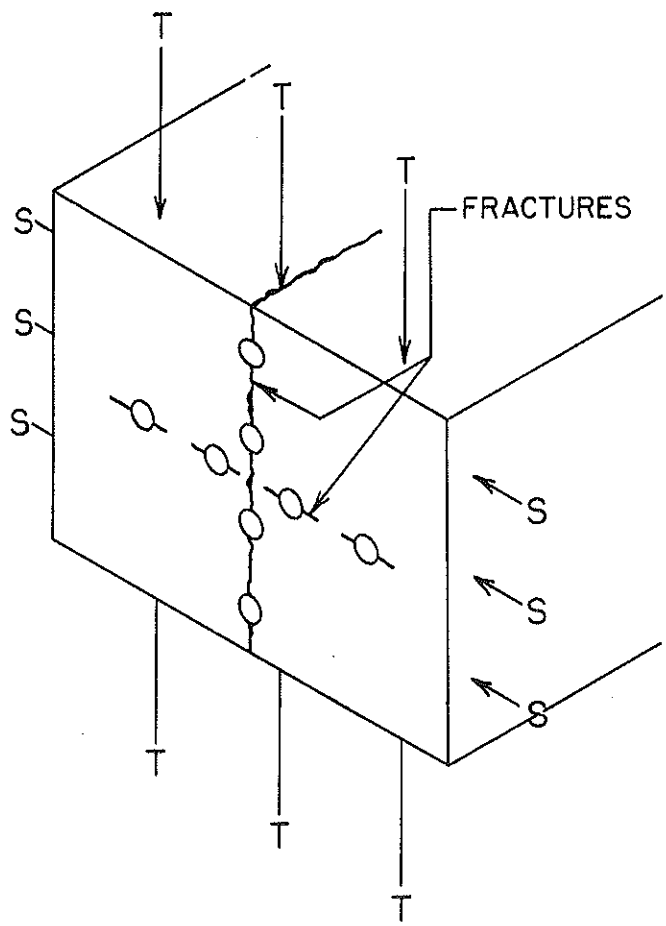


Figure 9. - The effect of a nonuniform stress field on the establishment of fractures between boreholes by pre-split or smooth-wall blasting.

Figure 10. - Blocky nature of the slope bank of a road cut in a jointed rock.

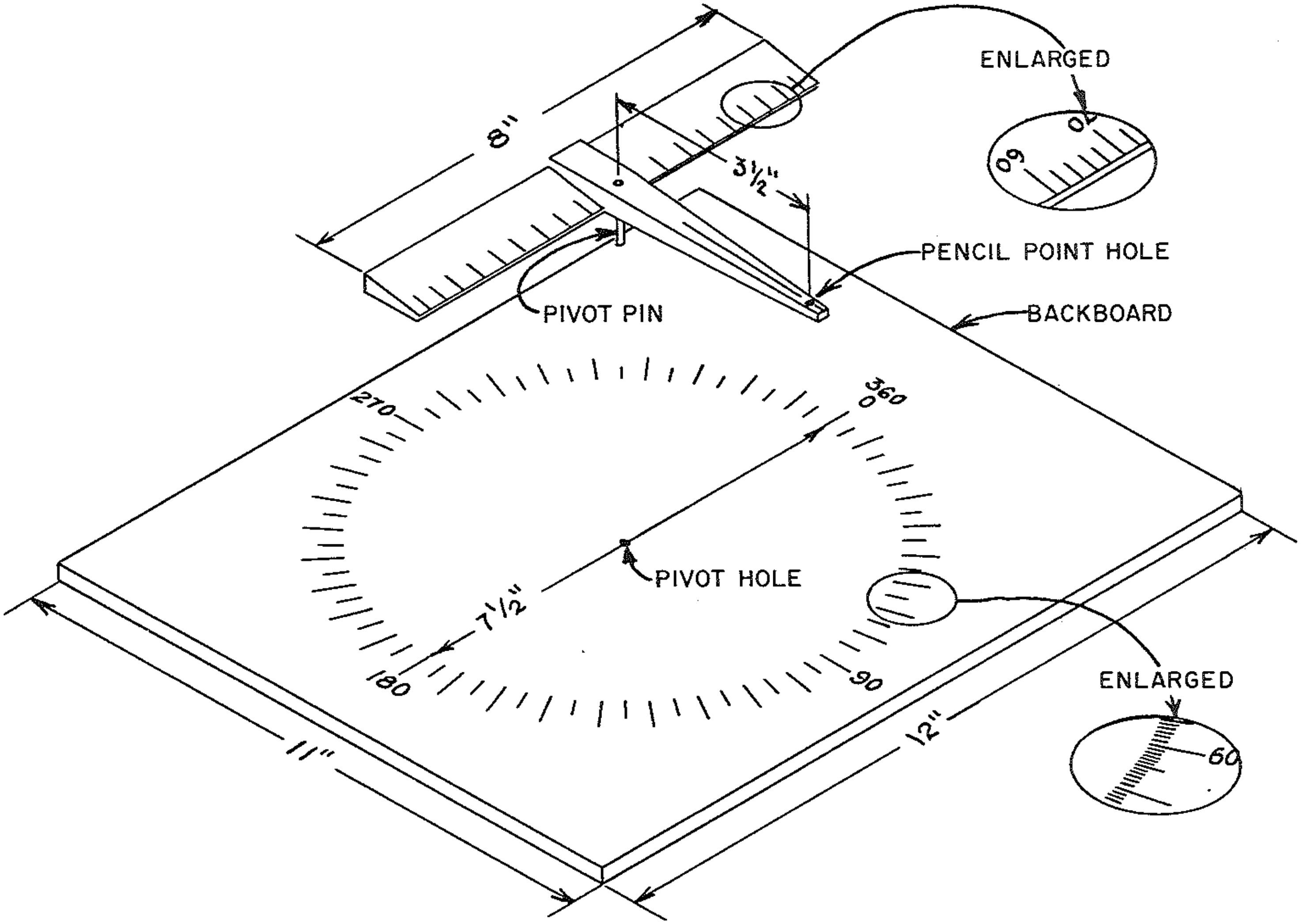


Figure 11. - Exploded isometric drawing showing details of the rock structure dip point plotting device developed by Bureau of Mines.

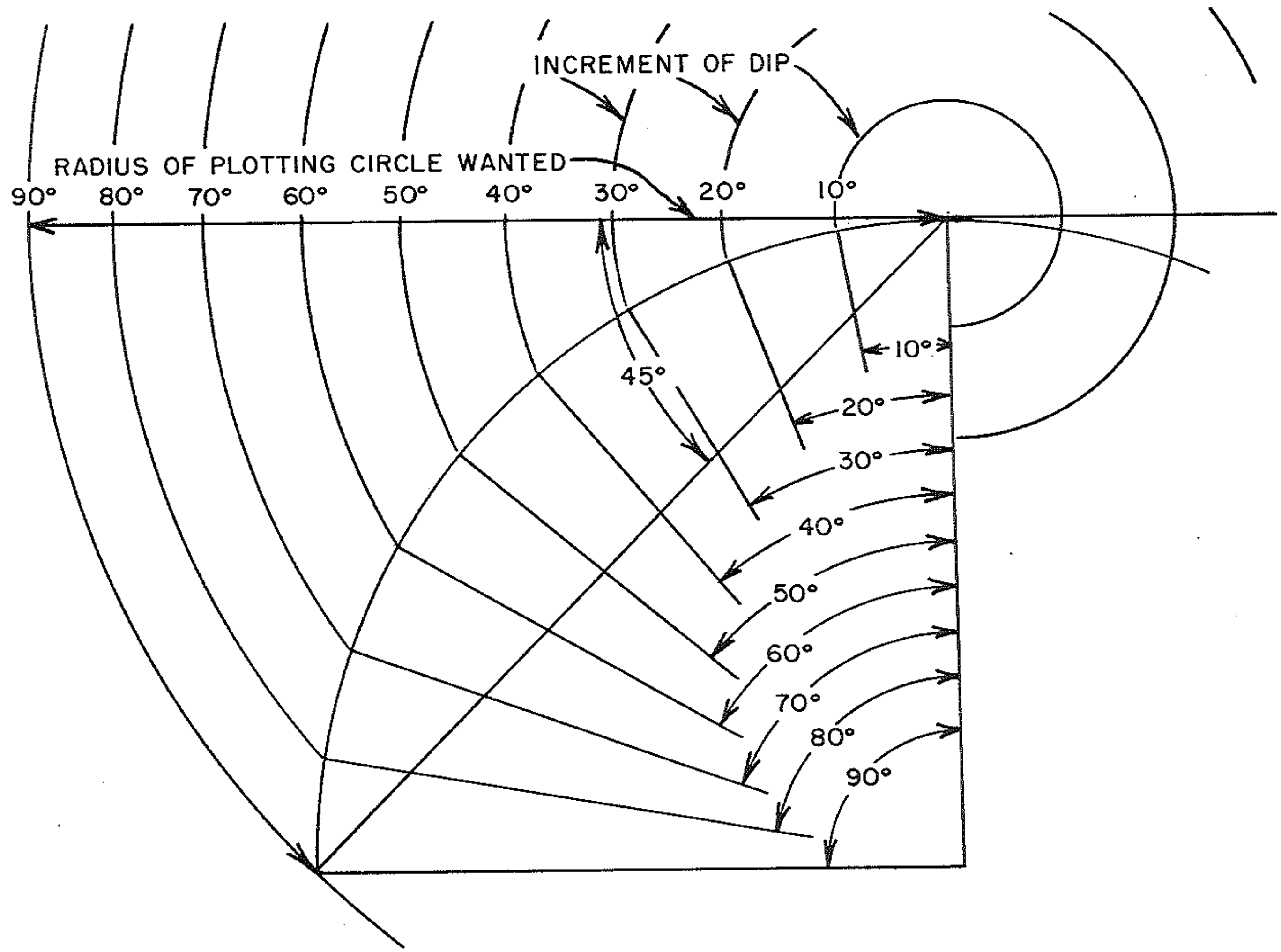


Figure 12. - Development of dip angle circle increments of a Lambert's equal area polar projection for any selected size, lower hemisphere plotting net.

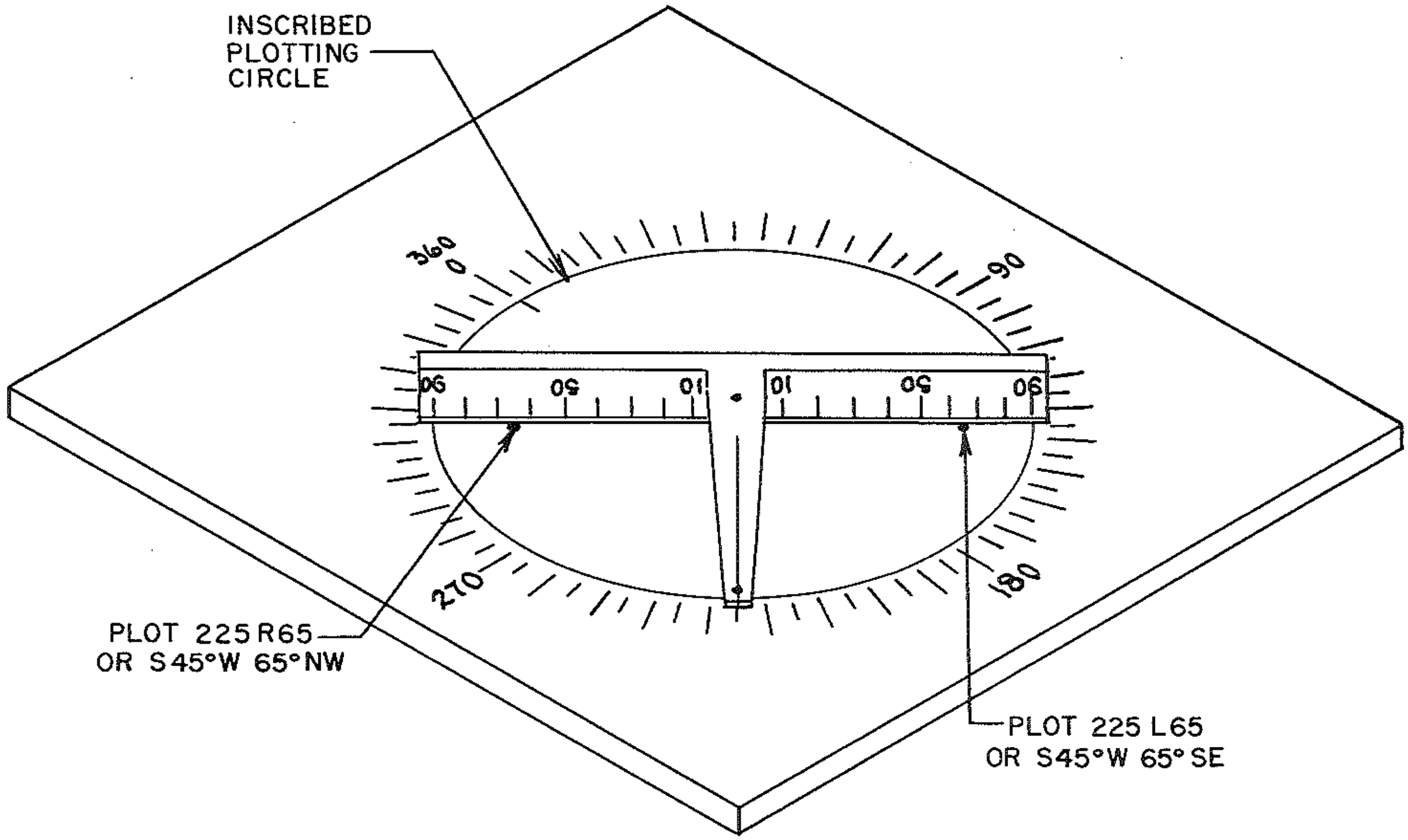


Figure 13. - Plotting the planar structure measurements 225 R65 and 225 L65 on a polar net using the Bureau plotting aid.

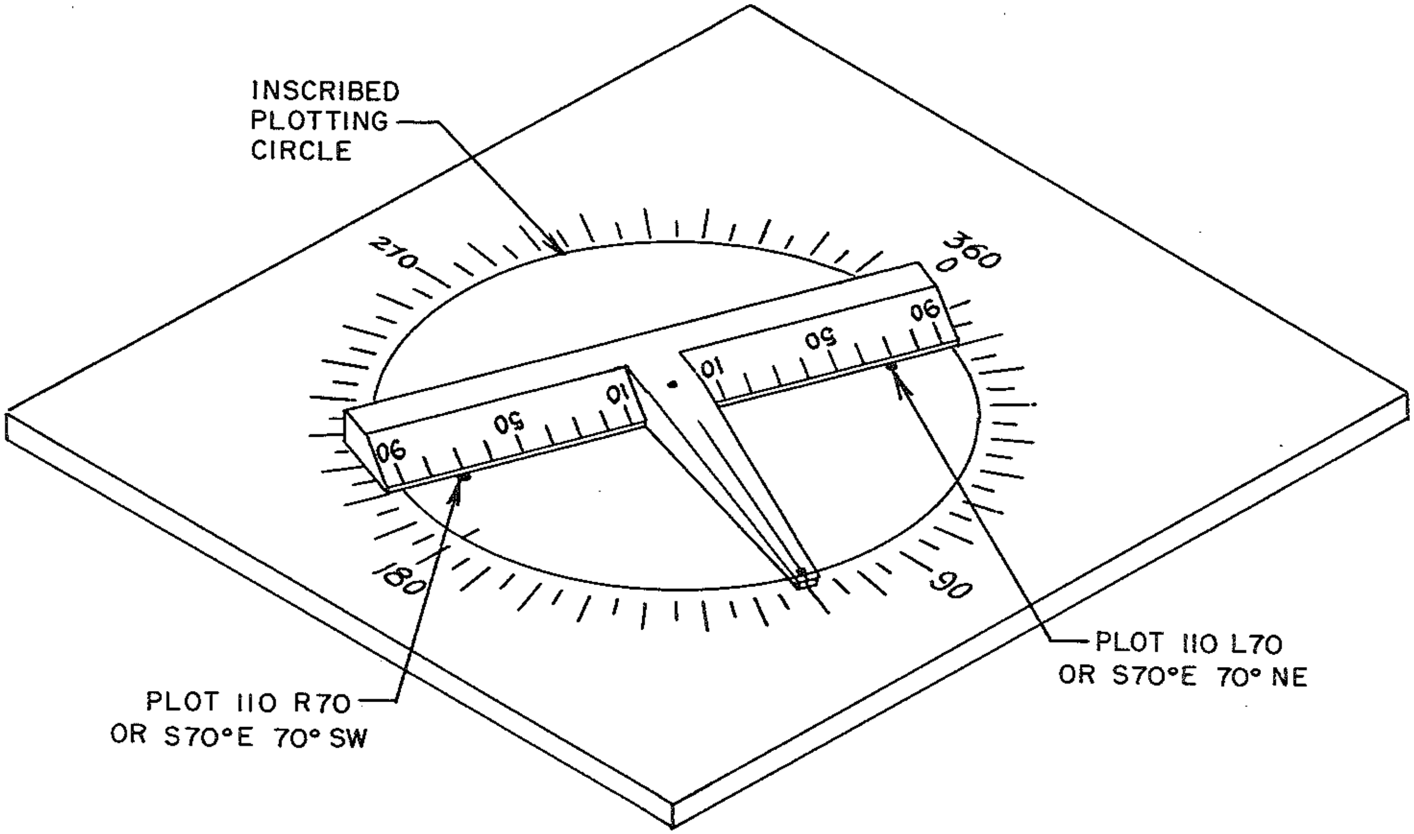
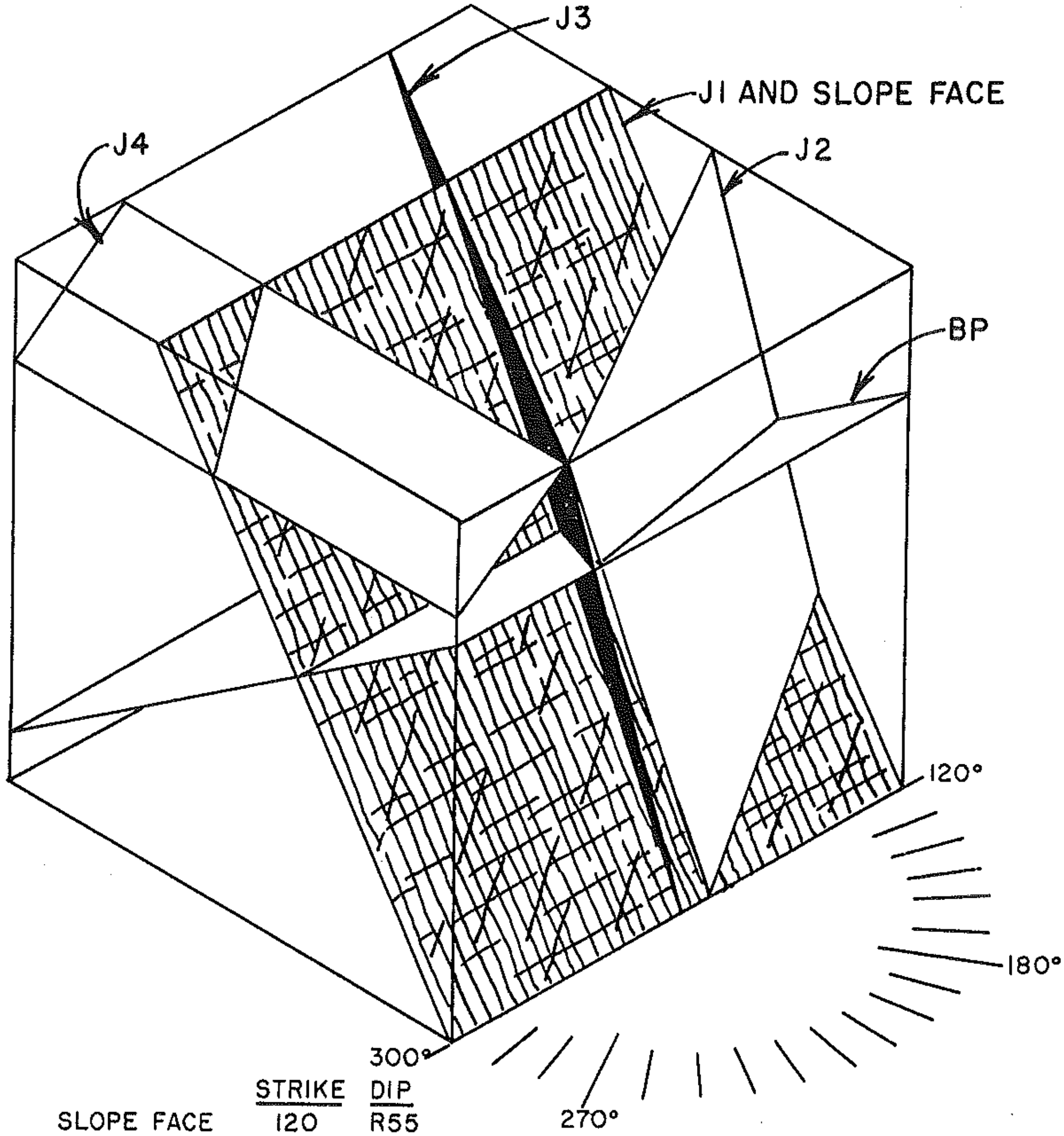


Figure 14. - Plotting the planar structure measurements 225 R70 and 110 L70 on a polar net using the Bureau plotting aid.



	STRIKE	DIP
SLOPE FACE	120	R55
J1	120	R55
J2	90	R84
J3	60	R70
J4	30	L45
BEDDING PLANE	120	L35
SLOPE HEIGHT	110 FEET	

Figure 15. - Block diagram showing the spatial orientation of joint and bedding planes relative to the angle of slope in the road cut illustrated in figure 10.

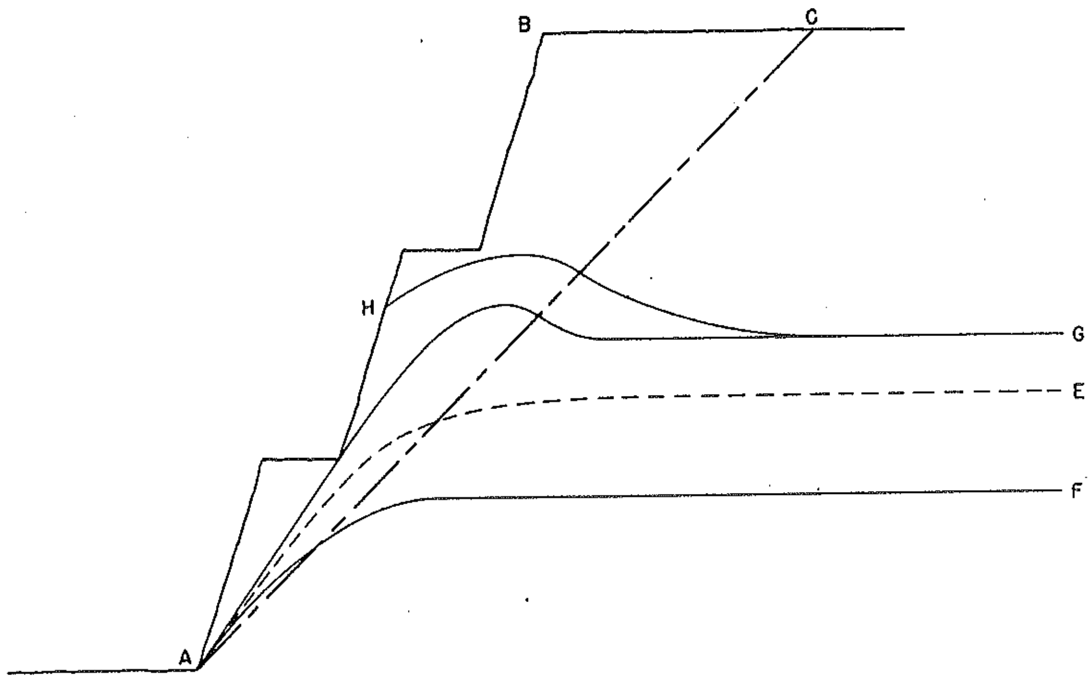


Figure 16. - Climatic-induced changes in configuration of water table in the slope wall of a road cut or open-pit mine.