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Stability of Rock Slopes

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Design of Rock Slopes

SHAILER S. PHILBRICK, Office of District Engineer, U. S. Corps of Engineers, Pittsburgh, Pa.

The design of rock slopes is discussed in this paper much as if an actual cut slope in rock were being designed. First, the engineering requirements of the cut; second, the geologic conditions of the site of the cut are established. These two define the problem. Then, using six basic principles, a generalized design to meet certain engineering and geologic conditions and the reasoning for it are presented.

In the design of cut slopes in rock, safety is emphasized because shear failures in rock are rare. For that reason the emphasis is on a cut slope that will yield very few rockfalls and will be structurally stable. In durable rocks the controlling factor in the angle of the slope, and hence the design of the cut, is the shape, geometric attitude, and stability of the rock segments to be exposed and activated on the cut slopes.

• IN THE DESIGN of rock slopes, the first step is to ascertain the purpose, depth, width, extent, geographic and topographic location, and orientation or azimuth of the cut. These are the engineering requirements of the cut.

What is the purpose of the cut? Or in connection with what kind of a project is the cut to be made? And in what stage of the project will the cut be made? For how long will it be open? Will it be over a restricted work area as adjacent to a bridge abutment? Who or what is going to be beneath or above the cut? If it is a channel change in massive granite in an isolated area where not even a stray fisherman will be at the foot of the cut, then slope design becomes of little concern as long as the hydraulic requirements of the channel are met. Conversely, if the cut is to supplant an existing tunnel on a turnpike or an Interstate route and is to be several hundred feet high and thousands of feet long, surely its design is a matter of great concern in regard to ultimate cost and safety.

Geographic location must be known because climate varies with geography. Hence, precipitation, which means slope saturation and necessity for drainage, can be anticipated. Depth and degree of weathering vary with climate. The number of cycles of freezing and thawing per year are tied to the climate. The topographic location affects the depth and degree of weathering, the movement of ground water, and the thickness and character of the overlying soil. The orientation of the cut slope fixes the exposure of the slope to the sun and therefore affects the number of cycles of freezing and thawing, and wetting and drying per year. It suggests the length of time that ice would form on the slope during the winter. Beyond the temperature effects of orientation are the equally fundamental relationships of centerline to geologic structure. Is the major joint system striking parallel and of great importance or normal to the centerline and therefore unimportant, or is it the common case of diagonal orientation to the centerline? In most cases it can be assumed that the data on the cut—centerline dimensions, purpose, etc.—are readily available and there is actually concern only with the characteristics of the rocks of the cut, or the geology of the cut.

Geology

The next basic step is the establishment of the type and characteristics of the bedrocks which must be shaped in the excavation to form a stable, enduring, safe slope

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that will cost a reasonable amount to produce and a minimum amount to maintain. Interest in the geology of these rocks is limited to those geologic characteristics and features that determine whether the slopes will be permanent; whether they will be structurally stable; and what the yield in rockfalls will be. Only those geologic characteristics, which affect the behavior of the rocks as slope-forming materials, are important.

The mineralogy of the rocks is pertinent to the point that the stability or instability of the minerals under the new conditions of stress and exposure is known and the minerals are recognized sufficiently to establish identity of the rock for contractual purposes. Also for contractural purposes, the top of rock, structure, relative hardness, degree of weathering, and similar properties of rocks need be determined.

The structure of the rock is required to define the probable size, shape, and orientation of the rock segments that will form the slope. Rock segment is the in-situ piece of rock that becomes the rockfall when it falls out of the slope. Knowledge of the structure of the rock is needed to define the attitudes and spacings of the planes that will cut the rock within and at the surface of the slope and form the rock segments. Thus, the centers of gravity of the rock segments and presence of sliding planes on which otherwise stable rock segments will move can be recognized before the slope is cut.

Geologic Investigations

The surface and subsurface investigations of the geology of the cut site should define the following:

1. Materials—rock and soil types, thickness, sequence, distribution;
2. Shear strength of the rocks;
3. Structure and tensional strength across and shearing strength along the structural planes;
4. Rock segments—size, shape, orientation, stability;
5. Depth of weathering:
   a. General disintegration—top of unweathered or sound rock slope;
   b. Along bedding, joints, fractures—top of inherent tensional and shearing strength of the structural planes;
6. Slope angles of these rocks in old or mature cuts;
7. Rate of weathering of these cuts; and
8. Water table and subsurface drainage.

The investigations should include both surface and subsurface conditions. The geological characteristics of the rocks as expressed in their outcrops and the lithology and structure of visible rocks are fully as much a part of the study as are the examination of the cores from the test borings.

The addition of geologic mapping may be somewhat difficult for those who are used to studying cuts mainly from the borings. However, it is far easier to determine from outcrops which are the important structural planes cutting a rock formation than from a series of borings. It is suggested that the borings themselves not only be directed toward establishment of the lithology and thickness of the several rock types but also be oriented to intersect the major joints and bedding so that the bounding planes of the rock segments can be determined before the beginning of slope design. The size, frequency, and method of drilling of the test borings are functions of the local geology and beyond the scope of this paper. Sufficient physical testing should be performed to establish the strength of the rocks where it is unknown.

The geological investigations should include a study of the old cuts in similar rocks in the area of the proposed cut. This study should find out the angles of slope that these rocks assume under prolonged exposure, and the rate of weathering of these rocks. The slope angles are readily recognizable and measurable. The rate of weathering or retreat of the cut slope may be measured in inches per year as on some of the indurated clays or compaction shales in the Pittsburgh area or a zero rate on some of the resistant limestones in Tennessee. The measurements may be based on comparison of the present position of a point on the slope with its former position using as a datum an adjacent structure built into the slope, such as a tunnel portal or a resistant layer whose outer edge was probably on the original cut surface. From these studies it may be learned that slope-protection devices (such as those installed on the Pennsylvania Railroad in eastern Ohio probably 50 years ago) are still entirely satisfactory. The stability of certain high cuts may be observed in which the shearing stress acting over many decades as a long-term load test has not caused the indurated clays or compaction shales to fail. In another cut, a failure may be found and thus may be perceived the order of magnitude of the strength of the stressed layer under field conditions or the effect of a different loading condition caused by a high water table or a dipping-joint system.

The findings should be recorded graphically:

1. The vertical and horizontal limits of the several rock types;
2. Base of mass rock disintegration;
3. Base of weathering along the planor elements, which is the approximate upper limit of any significant cross-plane or parallel plane strength in the rock;
4. Approximate, even if schematic, distribution of the planor and linear elements that will affect the stability of the rock segments; and
5. Water table and subsurface drainage.

The resulting drawing shows geologic data for the purpose of helping to solve an engineering problem. Therefore, the geologic attributes of the materials must be emphasized. The soils shown thereon may be identified under an engineering classification such as the Bureau of Public Roads or the Unified Soil Classification System of the Corps of Engineers. In addition, they must be described in such detail that the correlation with the unweathered or unaltered equivalents at other locations on the same section or on adjacent sections can be established without question. The typical slope angles of the weathered and unweathered materials to be encountered in the cut should be indicated. The rates of weathering of the several materials and the kinds of weathering products ought to be recorded. Finally, the approximate shearing strength of the materials should be noted for comparison with the estimated shearing stress.

PRINCIPLES OF SLOPE DESIGN

There are six basic principles to be considered in designing slopes:

1. Fitting the slope to the material, not the material to the slope;
2. Predetermining the shape and orientation and thus the stability of the rock segment that could become the rockfall;
3. Preforming the cut surfaces;
4. Protecting the cut surfaces;
5. Fitting the benches to the dangers; and
6. Draining the surface and subsurface waters.

Material and Slopes

The simple principle of fitting the slope to the material is one of the cardinal principles in dealing with soils, but when rocks are encountered this principle seems to be overlooked. This may possibly result from an engineering concept found at many drafting boards that all rocks are brothers under the skin and should be cut on ¼ to 1 and from an adjunct theory which holds that once a rock always a rock because rocks do not weather during the life of the cut. Both of these ideas are incorrect, and there are maintenance costs and maintenance men in many districts which can disprove them rapidly. There are all sorts of rocks and what is a good slope for one type in one situation may be an extremely poor slope for that same rock in another situation or a different rock in the same situation. Hence, the emphasis earlier on the geologic and site conditions and the behavior of those rocks in comparable cuts, because a table of slopes for all rocks under all conditions has not yet been developed. A start on this was made when such a study was conducted in the Pittsburgh area in connection with Youghiogheny Dam in 1941 and the results published in 1953 (3) and 1960 (4). In due course such studies should be conducted for other areas, but until they are available, field studies should be made to determine appropriate slopes for individual rock types.

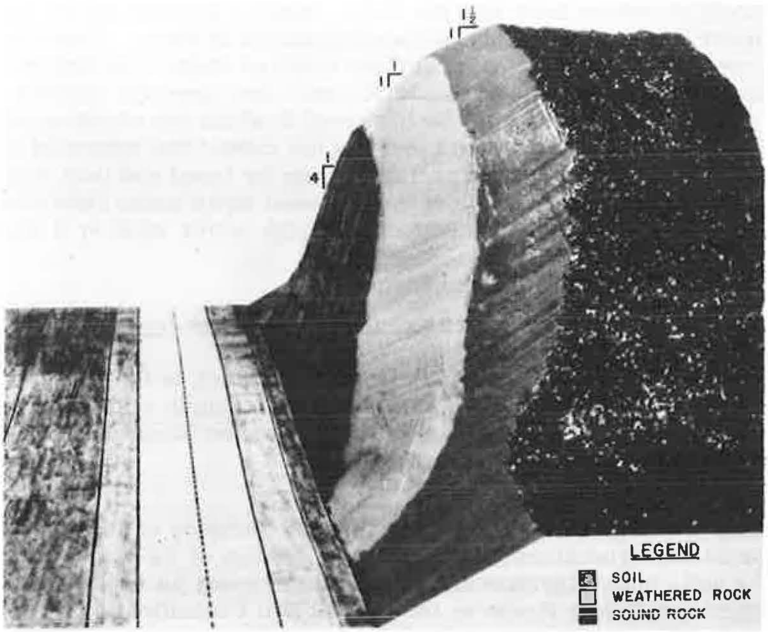

Figure 1. Oblique view, model of simple cut.

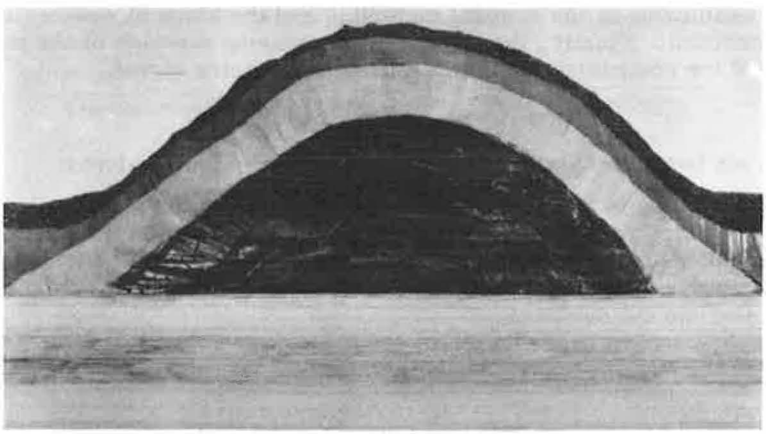

Figure 2. Front view, model of simple cut.

One of the most prevalent problems with material-controlled slopes is the simple three-layer condition of soil, weathered rock, and sound rock. This condition is most noticeable when the sound rock slope is ¼ to 1, the weathered rock slope is 1 to 1, and the soil is 1½ on 1. The effect of topography on the design of rock slopes is shown in Figure 1, which is a slightly oblique view of a model of a cut through a ridge composed of these three materials. The soil is gray colored with 1½ on 1 slope. The weathered rock is light gray with a 1 to 1 slope. The sound rock is dark gray in color with a slope of ¼ to 1. The black, white-flecked surface at the right of Figure 1 was a bright-green reflective surface representing grass on the model.

It can be seen from the front, as in Figure 2, that the sound rock core of the hill is mantled by weathered rock and that, in turn, by soil. It can also be seen that the ¼ to 1

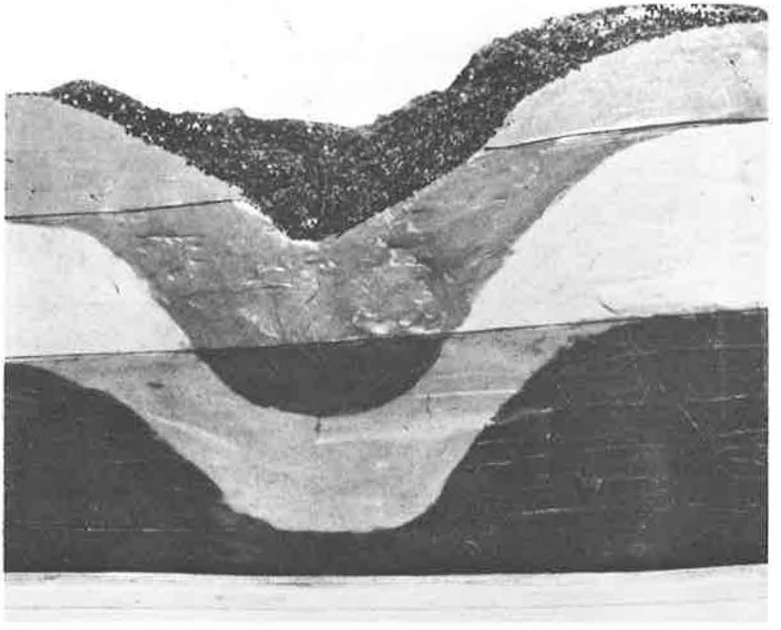

Figure 3. Front view, model of cut through ravine.

slope on the sound rock is mantled by 1 to 1 on the weathered rock and that, in turn, by $1\frac{1}{2}$ on 1 on the soil. The slope of the cut is dependent on the spot where the cut is made; that is, through the soil only, through the soil and weathered rock, or through all three materials. A uniform slope is not appropriate throughout the full length of this cut but must be fitted to the material at each station in the cut.

Another variable may be added to this last model and the crest of the ridge may be creased with an old ravine in which weathering has progressed to some depth. Figure 3 shows a replica of a design that fails to consider the effect of weathering and the change in materials with change in topography. The soil and weathered rock zones descend with the side slopes of the ravine and encroach into the slope zones established for the cut on the basis of conditions on the ridge crests on either side of the ravine. Such a design which develops 1 to 1 and even $\frac{1}{4}$ to 1 slopes on soil is going to be a nuisance if not a disaster to the traveler and the maintenance man. Therefore, the slope whould be matched to the material.

Another example of material-controlled slopes is shown in Figure 4, which is a geologic section through a double-track railroad cut. Two rock types occur in the cut, both of which show weathered as well as unweathered zones. The basal rock is a sandstone which was cut on a slope of $\frac{1}{4}$ to 1. The cap rock of the cut is likewise a sandstone but it is deeply weathered and therefore is cut on a flatter slope of 1 on 1. In between these two rocks is a clay shale, a very fine-grained but somewhat cemented rock, which requires a flat slope of 1 on 1 in both its weathered and unweathered expressions. The rock segments in this cut are random, unoriented, somewhat tabular, flat-lying shapes which are mainly stable with these cut slopes. Nevertheless, a narrow bench at track level was provided.

Rock Segments

The second principle in slope design is the predetermination of the shape, orientation, and stability of the rock segment that could become the rockfall.

Usually, individual grains in granular materials have an essentially random orientation. However, the chances are much greater in many rocks that the segments are oriented rather than unoriented. Because this is a matter of probabilities, it is profit-

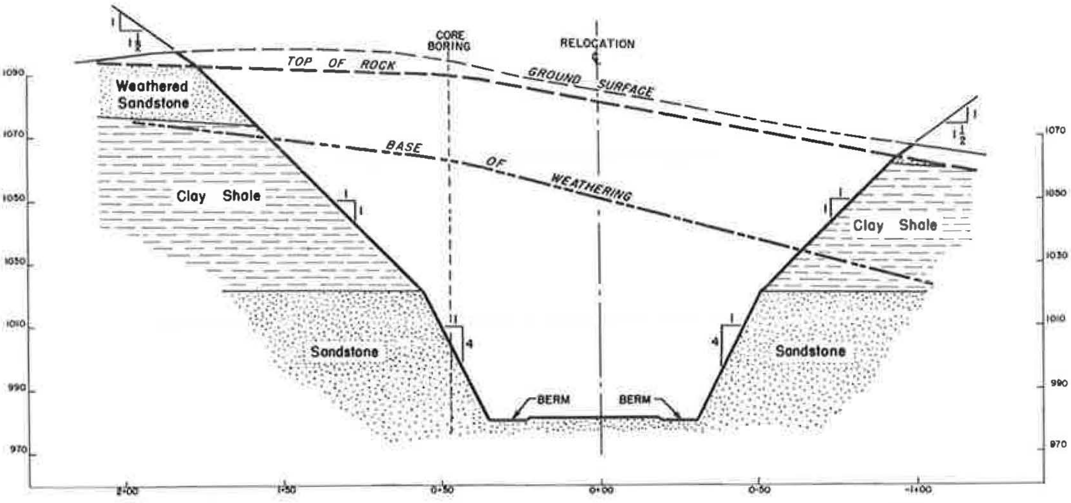

Figure 4. Through cut in rock.

able to examine the orientation of the rock segment. As long as the orientation is stable, the segment will remain in place in the rock slope as just another rock segment, but if it is unstable it will fall down on the road surface to become a menace.

The orientation of the rock segment depends on its shape and the attitude and spacing of its bounding planes. The attitude and spacing of sets of bounding planes can be established by common methods of study of surface and subsurface geology. If the outcrops in the vicinity of the proposed cut are of sufficient size and number, then the planes can be identified and their attitudes and spacings observed and measured so that a reasonably reliable, although not a statistically reliable, estimate may be prepared. If rock exposures are missing or rare, then normal subsurface investigation must be supplemented by directional drilling to hit the planes being investigated.

Identification of these planes as geological features rather than just as geometrical features bounding geometric shapes will hasten the recognition of the probable shapes of the rock segments. In the sedimentary rocks, the most prominent planes are the bedding planes. Because sedimentary rocks underlie most of the region from the Piedmont to the Rockies, bedding planes may be the most important single structural feature in the rock and fortunately one of the most easily recognizable. With bedding planes forming the top and bottom of horizontal segments in horizontally bedded rocks, then the other bounding planes would be identified usually as joints of one sort or another, generally nearly normal to each other and the bedding planes. By inspection it appears that the general case in segment shape in sedimentary rocks is a slab varying between a paper-thin sheet (as in a fissile shale) through a thicker slab in the massive sandstones and limestones to a cubic form in coal. The orientation and, hence, the stability of these segments will depend on the local geological structure at the site of the cut.

In the metamorphic rocks the bedding planes of the sediments have a counterpart in the foliation and cleavage planes of the schists and gneisses and slates, although the probability is that these planes are not as weak as the bedding planes. In the igneous rocks, original flowage, crystallization, cooling, and shrinkage structures provide planor elements which shape the rock segment. They are much less easily recognizable, except in the lavas, than the bedding planes in the sedimentaries. Other surfaces of the rock segment are formed by joints of one sort or another.

The original segment shape, regardless of the rock type, may have been changed by later processes which formed other sets of planes to cut the segment into smaller pieces with different shapes. Because of the different attitudes of these later planes the present shapes may not be stable on the slope which was stable for the original

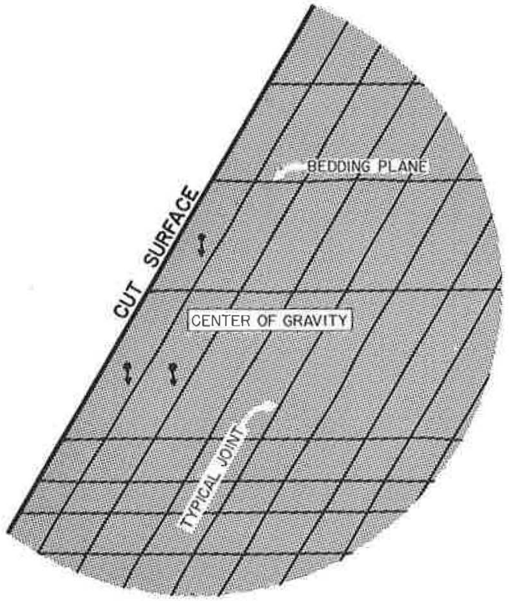

Figure 5. Stable rock segments.

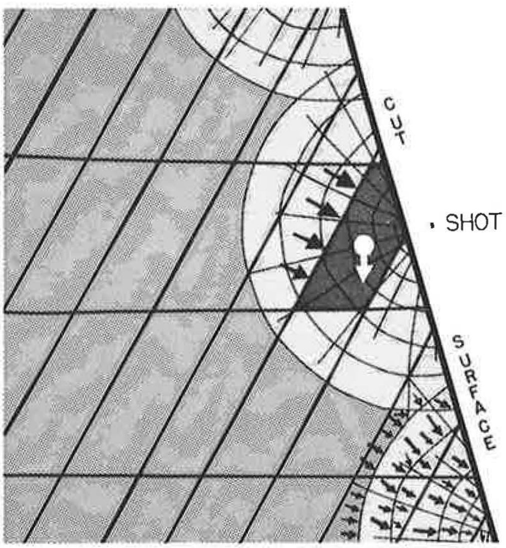

Figure 6. Potentially unstable rock segments.

segment shape. These later planes can be expansion joints parallel to hill slopes developing with the release of strain. They may be less complicated fractures caused by freezing and thawing. They may be tension cracks caused by cantilevering of beds above, when underlying strata were eroded along a slope. But these later planes always tend to complicate the pattern of the rock segments in the slope.

There is another structural feature which shapes rock segments—faulting. Rock segments adjacent to the fault as well as within the walls of the fault are affected. The feather joints trending out of the fault create a new system of fractures and new shapes of somewhat smaller size than the original rock particles. But within the fault zone itself the reduction in size may have proceeded to grains of sand size in the siliceous rocks and to clays in the argillaceous and calcareous rocks. Thus, the problem of slope design in rock, which must consider the size of the rock segment in the slope, may suddenly take on the aspects of a problem in soils, even if only for short distances.

Figure 5 shows a horizontally bedded rock cut by a series of joints dipping 60^0 to the left. The center of gravity of several rock segments near the cut face is shown with vertical arrows. These segments can be seen to be stable.

Figure 6 shows the same conditions but the site is on the left wall of the cut; therefore, the segments appear to lean into the cut. The apparent forces are shown by arrows against the side of a segment that is supported by a small triangular segment at the cut surface. As long as the cut-side segments remain in place, the rock mass is stable; but if those segments move or fall out, then the landward segments will topple and rockfall will result.

Preforming the Cut Surface

Much good design has been downgraded by poor construction methods—notably by overshooting of finished slopes with its resulting fracturing of remaining rock and production of material for rockfalls that did not exist at the time of design. Not only does the overshot rock fall down the slope, but the resulting face is so rough and broken that weathering is accelerated and high frequency of rockfalls continues long after it should have declined.

Figure 7 is a modification of Figure 6 and shows the effects of blasting put off at a point labeled "shot" located within the cut area. The original segments have been shattered into odd-shaped fragments within the blast areas by radial and circumferential cracks until the original segments no longer exist and new segments with new conditions of stability or instability occupy the sites of the original segments. The motion of these fragmental segments as they move sooner or later into the cut is shown by the black arrows. Examination of the original segments, heavily outlined, shows that slope-side buttress segments are now shattered and ready to fall out. The pressure arrows of the landward, unstable fragments will soon force the fractured segments over the slope to be followed by others as rockfalls that will continue for a long time.

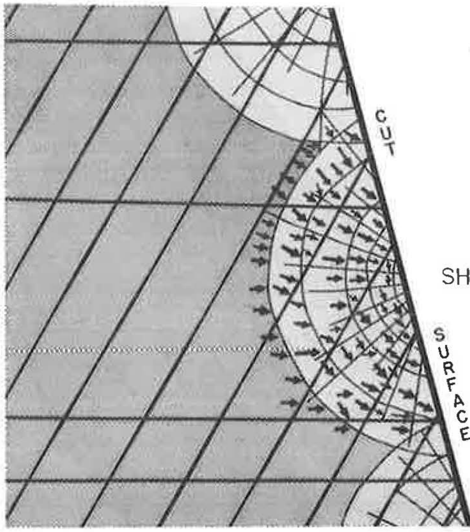

Figure 7. Segments shattered by blasting.

For many years, it has been standard practice to preform the sides of excavation where the concrete is to be placed against rock by closely spaced drill holes along the line of the excavation. In earlier days, the spaces between the holes were broken by broaching so that an actual crack was formed along the limiting line of the excavation. When the rock was shot, the break line usually followed the line of the drilling and broaching. This procedure has been much simplified by a process called presplitting which was developed to a high degree on the Niagara Power project, although it had been used nearly 30 years before. This procedure was used on the Southern Railway (ENR 5) as well as at Opekiska Lock, Monongahela River, W. Va. The limit line of the excavation if formed by drill holes on centers up to 4 ft in which small-stemmed charges of dynamite are suspended from Primacord at suitable vertical intervals in each hole and the charges detonated to crack the rock in a line between the drill holes. The resulting free surface effectively limits the disturbance of the wall rock during blasting to such a degree that overbreak is negligible.

This method has been used in limestones, some of which carried shale partings. In the construction work of the Pittsburgh District, Corps of Engineers, presplitting has been used successfully in typical Conemaugh formation sandstone (a subgraywacke) and silt shale. Offsets and interior corners have been presplit and have held their shapes at Opekiska Lock.

Presplitting of cut slopes will reduce the disturbance of the slope-forming materials and reduce the quantity of rockfalls which will, in turn, decrease the maintenance costs. The reduction in rockfalls will also decrease the quantity of scaling required during construction and tend to offset the small cost of drilling and shooting to presplit the rock. It is believed that some positive measures must be taken to reduce the shattering of the slope rock which is one of the major causes of rockfalls. Preservation of the rock segments in their preconstruction condition permits design assumptions to be made and plans prepared without fear that construction procedures will invalidate them.

Protection of Cut Surface

Slopes may be structurally adequate, or almost so, if protected from weathering and deterioration. Like anything else in this work, the desirability of protecting slopes can be established on engineering and economic principles and should be undertaken on that basis. Protection of the cut surfaces includes everything from self-protection (using the in-situ materials to protect the surface) to construction of protective structures.

Here are three examples of self-protection. With rapidly weathering rocks, such as the indurated clays and clay shales, slopes may be protected by using an angle sufficiently flat to permit the retention of the insulating blanket of fine weathering products which soon forms and seeding such slope as the blanket develops. On the other hand, steep slopes in durable rock composed of large segments may be protected by holding the outer layer of segments in place by rock bolts set in grout. Instead of paving the benches that needed protection, a resistant bed has been used as the bench surface or the bench has been located immediately above a resistant bed.

Midway between self-protection and structural protection lies the area of surface protection. Here, coatings and meshes are used with and without rock bolts to secure the surface treatment to the rock mass. Surface protection against moisture loss and weathering may be done with bituminous coatings or portland cement bearing coatings. The life of these coatings may be as much as 25 years under rigorous conditions of weathering.

Structural protection usually in the form of slope pavement or support of overhanging ledges resulting from overbreakage or differential weathering may be far less expensive than additional excavation, particularly in areas of high relief and expensive real estate.

Last within this area of slope design is the protection of the rock surface from loss of moisture by directing the flow of available water over those rocks that hold their strengths only as long as they hold their moisture.

Fitting Benches to Dangers

Benches used in the design of rock slopes should be considered as rock catchers. They are not primarily access roads to some higher cliff nor are they devices to reduce the loading on the toe of slope. Benches are to catch rocks, not soil slides, from the overburden which can readily fill the benches and destroy their usefulness as rock catchers. Soil slides should be prevented by adequate soil design and protective planting.

If benches are to fulfill their function, they should have the following characteristics:

1. They should be located where they can catch rocks, preferably at base of the rockfall producing zones.
2. They should be permanent.
3. They should be accessible but the feature of accessibility should not be permitted to determine the location of the bench.

There is no basic reason why access to benches must be accomplished only within the limits of the cuts or why benches need be interconnected to the detriment of their rock-catching ability. Access roads to benches can be located outside of the limits of cuts in many places, thus keeping the bench snug beneath the rockfall zone and reducing the chance for the rockfall to skip over the bench and proceed down the slope to the roadway.

Drainage

The normal practice of diverting surface water from the face of the cut is excellent except in the rare case where moisture retention in the compaction rocks is necessary. Following the basic principle of diversion, water collecting on benches would be drained toward the hill and thence laterally to outfall drains within the cut limits or to the ends of the cuts.

Subsurface water unless drained through natural watercourses (such as fractures, joints, or bedding planes) can build up hydrostatic pressures which have in times past caused shear failures in rock slopes. To prevent this, the Corps of Engineers in 1943 drilled horizontal drain holes to depths of 250 ft beneath the top of the 300-ft deep cut at the Youghiogheny Reservoir spillway at Confluence, Pa. Admittedly subsurface drainage of rock slopes is rare, but it should be considered and installed where slope failure would result if hydrostatic pressure of critical magnitude is probable.

SLOPE DESIGN

After identifying the engineering considerations and the geological conditions, a rock slope may be designed using the principles that have just been reviewed. To make the design process a little easier rock slopes may be classified on the basis of the durability of the rocks composing the slopes. This would give three types of slopes:

1. On durable rock;
2. On nondurable rock; and
3. On combinations of durable and nondurable rock.

There is a final consideration. If the rocks are obviously far stronger than required

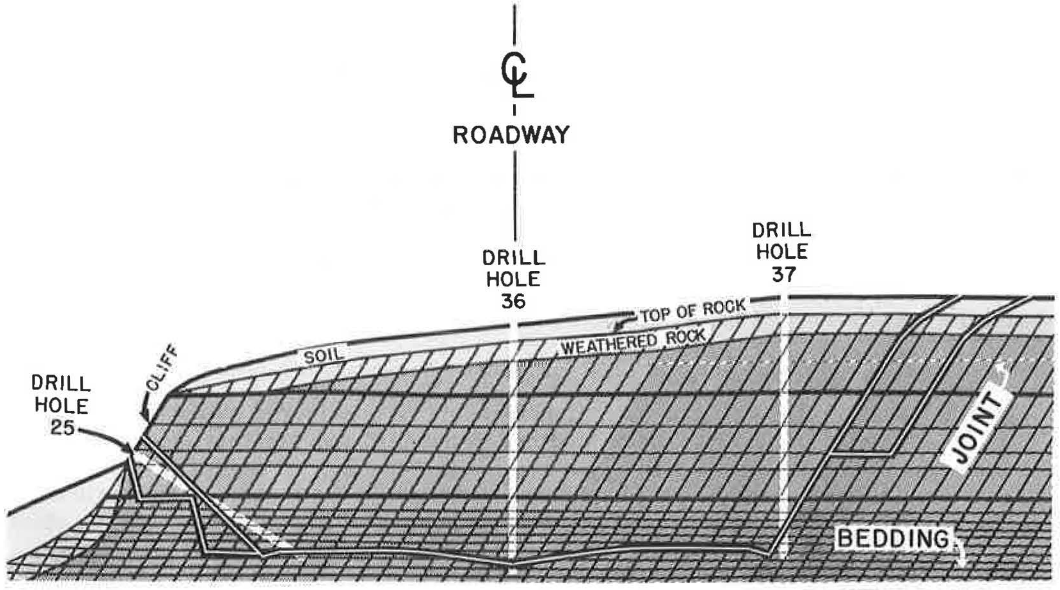

Figure 8. Geologic section of cut.

by the estimate of the expected shearing stress of the slope and the rocks will remain of relatively the same strength and not disintegrate during the life of the slope, then the shearing strength does not control the over-all slope angle. If, however, the shearing strength closely approaches or is less than the estimated shearing stress, then the slope must be redesigned against a shear failure.

Durable Rocks

In the strong rocks, the slope should be designed as steep as the geologic structure of the rocks and the stability of the rock segments will permit. The upper limit would be that where the cut was to be made entirely within the limits of one rock segment. This is possible in a massive rock free of all bedding and joints. Then the cut would be made vertical or even overhanging. If a choice is made to design, under normal geologic conditions, steeper than the geologic structure permits, the rock segments will become unstable and rockfalls will result, requiring increased maintenance. The oversteepening should be limited so that the increased cost of maintenance does not exceed the construction savings expressed as an annual charge composed of the items of amortization and interest over the life of the slope.

In considering the costs of increased maintenance vs increased construction costs, liability should not be overlooked. The courts have a tendency to examine the causes of landslides to ascertain whether they are natural phenomena or the results of actions by individuals, corporations, or public bodies. It seems that the trend is to hold the public construction agencies responsible for negligence. Thus the savings created by reduced construction costs must be weighed not only against increased maintenance costs but also against a possible catastrophe where proof of negligence would wipe out the construction savings in a moment.

Figure 8 shows a geologic section of a cut with a roadway. The problem is to construct the slope rising from the back side of the ditch to the ground surface in such a way that both construction and maintenance costs are at a minimum and the road is safe to drive at the design speed at all times. Now comes the need for all the information from each of those borings which seemed so numerous when they were planned and now seem so few.

From the top of rock down to the base of primary weathering where the rock becomes truly rock-like in character—unweathered and hard—the slope angle should be designed on the basis of the shearing strength and resistance to erosion of the material. A slope of not steeper than 1:1 would be appropriate. In the unweathered rock below, the slope angle that would develop from a vertical cut would be dependent on the orientation, nature, and behavior of the rock segments composing the slope-forming rock. The section shows that the grain or rift of the rocks, represented by the joints, dips to the left at an angle of about 60^0 with the horizontal. This is a convenient and not unusual angle for a slope and so it is chosen for the right side of the cut. The rock segments in that area of the cut are defined by the attitude of the bedding and the joints. The second joint which cannot be represented on the section is in the plane of the section and serves to form a rhomboidal rock segment. By inspection, it is clear that the segment will not fall into the cut. Thus, the right side of the cut is stable when cut on the joints as shown in Figure 5.

On the other side of the cut, the joints dip into the slope, as shown in Figure 7. In such a situation the common methods of blasting will leave a jagged surface partially on the overhanging joints (the underside of the segments) and partially across the joints; that is, through the segments. The stability of the fractured segments remaining adjacent to the left slope surface depends on where the cut slope passes through the fragment and how much of the next roadward fragment remains as a buttress. The rock segment is a high, slabby piece of rock with a narrow base which does not underlie the center of gravity of the segment and is unstable. Most assuredly, a large number of the fractured segments are going to come falling down on the roadbed as rockfalls unless something is done to hold them in place, catch them, or cut the slope in such a way that such shapes are not formed. It would seem wise to presplit the left slope.

Catching the rockfalls is quite acceptable unless the room required for the benches equals or exceeds the width required to flatten the slope to the point where the segments are stable and rockfalls cease to occur. In this case less excavation would be required by a 1 on 1 slope than by a steeper slope and a bench.

For many persons, a bench on a rock slope, or at its base, is a necessity as an evidence of good practice and an insurance feature. The width of the bench is dependent on what kind of equipment is to be used to clean up the bench during maintenance. The spacing of the bench above the roadway is a matter of individual choice in most cases governed to some degree by the vertical reach of the maintenance equipment to be used in scaling the slopes for the prevention of rockfalls. Thus, the 40-ft maximum height suggested by Baker and Marshall (2) has a rational basis provided the location of the bench meets the criteria already set forth.

Figure 8 shows a bench at the base of the thick-bedded rock on the right side of the cut, placed mainly from habit, although it is believed that the rockfalls on this side would be of the rock-slide type with less tendency to fall free and bounce laterally. Were there a request to add a bottom bench, then the upper bench would be unnecessary. However, on the left side where the rock segments are going to strike rough surfaces during their descent and bounce in random directions, it seems wise to interrupt their descent with an intermediate bench as well as the widening of the roadway with a bench at that level. In doing this, it is found that a stable 1:1 slope involves less excavation, so benches are actually eliminated on the left side.

Nondurable Rocks

Nondurable rocks are rock-like in character in place but will disintegrate during the proposed life of the cut to soil-like materials, the depth and degree of disintegration being dependent on the original material and slope design. These are the materials that Mead called the "compaction shales" and others have called "immature shales"; these lie just beyond the realm of "stiff clays." If formation names are applied, the list would include familiar names like Bearpaw, Cucaracha, and Pierre, and less familiar names like Oaks, Pittsburgh reds, and Dunkard red shales. Yet these are mappable, geologic formations with specific lithologies, faunas in some cases, geologic structure and ages in the millions of years. By all standards except durability and hardness, they are rocks. Even during excavation they behave as rocks and look like rocks. Slopes to be cut on them are governed by preexisting planes of weakness; in some cases, faults, and in other cases, shear planes resulting from regional deformation as well as the shearing strength of the rock. They are only briefly mentioned here because they will be discussed in detail in succeeding papers.

The design of the cut in the nondurable rocks must consider the eventual as well as the present character of the materials and should endeavor to preserve whatever strength these materials had before the cut. As the rock weathers, a flat slope will permit the formation of an insulating debris blanket and a weathered zone that will protect the underlying materials. If the blanket is absent and the slope steep, the entire mass may continue to weather to soil-like materials and the shearing stresses exceed the reduced shearing strengths. The slope angle will be controlled by erosion if the underlying rock mass retains its original strength. Preexisting, preslope zones of weakness will tend to control stability as with the durable rocks unless the nondurable rock is weaker than the shearing stress, which will then control the slope angle. Slopes on the nondurable rocks should be designed flat enough to prevent loss of the protective blanket on the surface as well as to prevent shear failure through the rock mass.

Durable and Nondurable Rocks in Same Cut

Designing with durable and nondurable rocks in the same cut is the typical situation in the Allegheny Plateau where durable sandstones are commonly underlain and overlain by less durable shales and clays. Here cuts are plagued with rockfalls coming from rocks in which there is rarely, except in the coal, a well-developed pattern of joints or fractures and the segments have a random orientation and distribution. Here, the drilling seeks to establish the sequence, thickness, lithology, depth of weathering, structure, and location of the beds that would form the slopes and the position of the water table. Because of the lack of a well-defined joint system, directional drilling would be employed nearer the hillside where expansion joints might have developed than in the middle of a through-cut area where only regional jointing would be expected. The water table studies would be directed to determining whether natural drainage would be sufficient to keep the slope mass drained or whether additional drainage by horizontal drains or tunnels would be required.

The author's (3) position on design of rock cuts in these materials 200 to 300 feet deep and now up to 20 years old was stated in 1953 and restated in 1960. Ackenheil discussed one of the really difficult slope designs in these rocks in a paper on the Fort Pitt Tunnel at Pittsburgh, Pa., (1). The following are the basic concepts:

1. Cutting each material to fit its characteristic weathering slope;
2. Addition of berms at the base of the rockfall producing zones;
3. Providing drainage to reduce the hydrostatic head behind the slope; and
4. Designing the over-all slope to be stable against shear failure.

ACKNOWLEDGMENTS

Harry F. Ferguson has provided constructive geologic criticism. Frank Pehr constructed the models and drawings in which he was assisted by Frank Tomasic. A. C. Ackenheil has critically reviewed the early drafts of this paper.

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Evaluation of Rockfall and Its Control

ARTHUR M. RITCHIE, Washington State Highway Commission

In the face of the requirements of "practical stability" and "comparative safety," the design phase of a rock cut is an ever-pressing problem. The situation is difficult because the fundamental relationships are complex and cannot be readily translated into a basis for design.

This study deals with the mechanics of rockfall from cliffs and talus slopes and discusses various ditch sections and rock fences that proved most effective in containing it. A mathematical consideration of rock trajectory is presented.

An intensive statewide study was initiated to determine what actually happens to a rock when it falls. Examples of falls were recorded in slow motion on 16-mm color film. Field work was conducted under varying conditions found in State-owned quarries and on existing highways. Natural and man-made talus slopes were also used.

It is concluded that the present standards dealing with rockfall design are inadequate, unrealistic, and even dangerous. Considerable expenditures are being made in an unsuccessful attempt to design for rockfall; and such expenditures have not resulted in safer roads. Design criteria describing a more realistic relationship among the variables of cliffs, angle of slopes, depth of ditches, and width of fallout areas are presented.

• ROCKFALL has been a plague to the design engineer and continues to be one because modern requirements demand high rock cuts. Often, funds are not available to cope properly with the problem. This fact has stimulated many papers in an attempt to find an economical solution. Many factors have been studied and evaluated, but in spite of this previous work, few engineers today are willing to say that this design will stop all rocks from falling, or that design is safe. It is a common experience that long after construction has been completed, many portions of a rock cut may produce a sustained quantity of rockfall. The familiar sign "Watch for Falling Rocks" is also visual testimony to the continued existence of the problem. Fallen rocks must be kept off modern highways.

This paper deals with rockfall in such a manner that any rock that falls, incidental or continuous, can be contained, regardless of the angle of slope from which it comes. The approach is novel and rather radical, considering existing design standards; yet the new approach is of proven practical value and embodies the maximum safety possible even though it may create, at the same time, another problem of "comparative safety." From the research work presently completed, rocks falling from cliffs can be contained by a deep ditch or a combination of a deep ditch with a special rock fence, whereas rock roll (what occurs on talus slopes) may require only a rock fence in conjunction with a normal ditch.

THE PROBLEM

One reason that current standards are inadequate and proving ineffective is that the problem has not been fully understood. It is doubtful that anyone has made a project of watching rocks fall before. It has been generally assumed that there is a direct rela-

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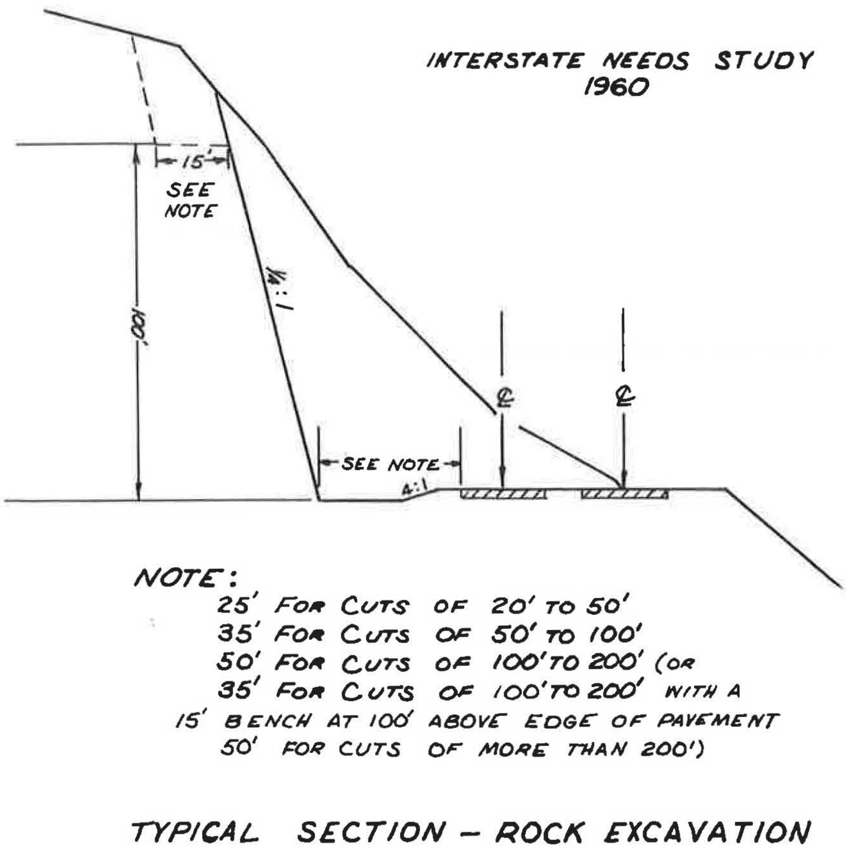


Figure 1. Current design standard.

tionship between the height of cliff and width of fallout area necessary to contain the material (Fig. 1). Figures 2 and 3 show that this relationship is in error. Present assigned values are arbitrary and unrealistic; hence, they are an unsafe solution. Another reason why the problem exists is that it is twofold in nature. Rockfall must take into consideration not only the minimum width of fallout zone determined by the frequency of stones making impact at a maximum distance from the base of the cliff, but also a means of stopping the stones after they have picked up angular velocity on impact. The latter feature particularly has not been emphasized enough.

Figure 4 shows a stone that is falling with but little angular rotation while in flight. After it makes impact, and especially if it strikes some inclined surface, it will begin to spin with tremendous speed. If the stone imbeds itself in debris on impact, much of this spin is dampened.

There is yet another reason why rock slopes continue to be particularly troublesome. The standard approach to the rockfall problem has been one of trying to restrain rocks from falling. This method does not consider or treat the case where a rock does fall. Most of the effort has been directed toward cutting more benches, flattening the slope, pinning down loose rocks, covering the area with wire mesh, etc. (Fig. 5). Actually, the choice of which restraining technique to use is, itself, so interdependent with other factors that at best it is an arbitrary solution. Too often the real answer is quickly covered by an avalanche of unruly factors that seldom go together neatly—and it is doubtful that attempts to restrain rocks from falling will ever develop into the proper approach for controlling rock slopes.

Certainly there has been an ever-growing need for a better method of predicting the state of stress in a rock slope or of discovering criteria for evaluating the many factors

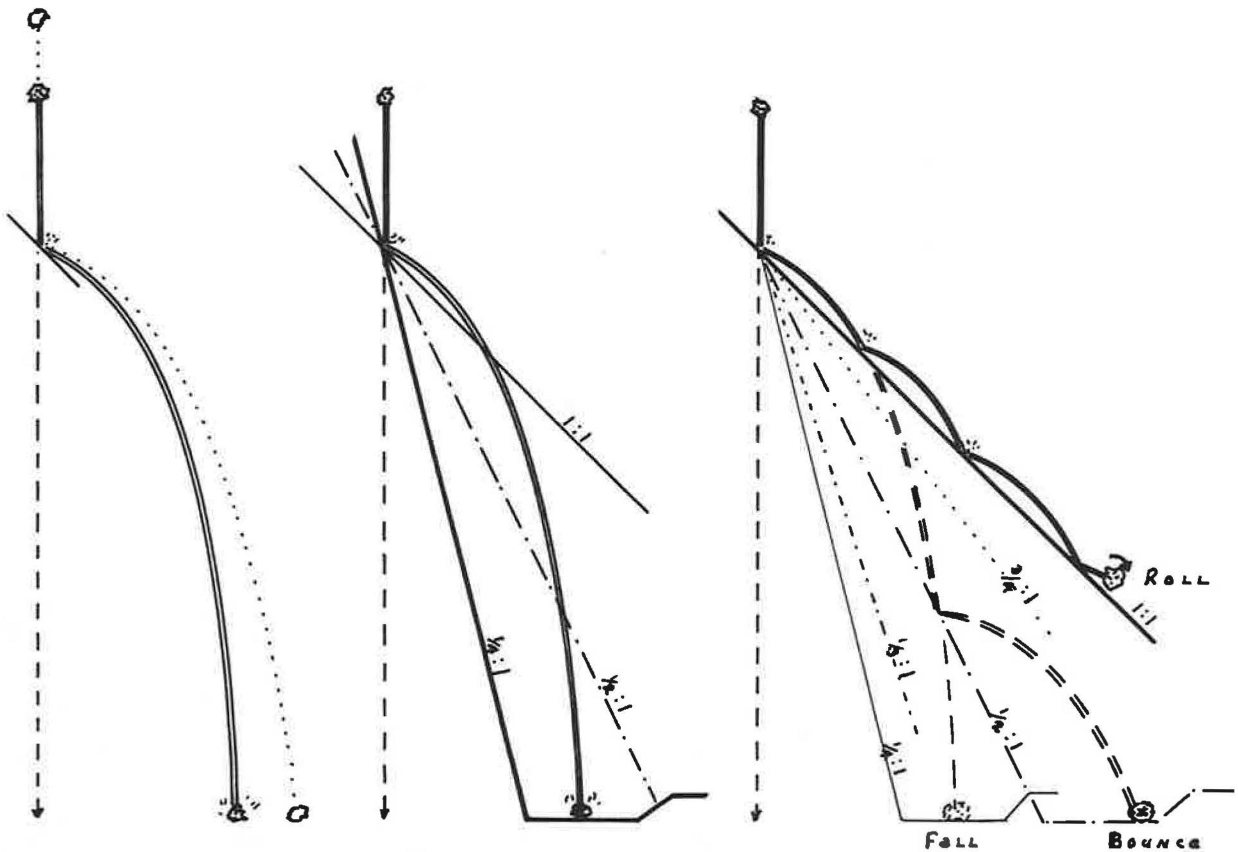


Figure 2. Path of rock trajectory superimposed on variable slopes.

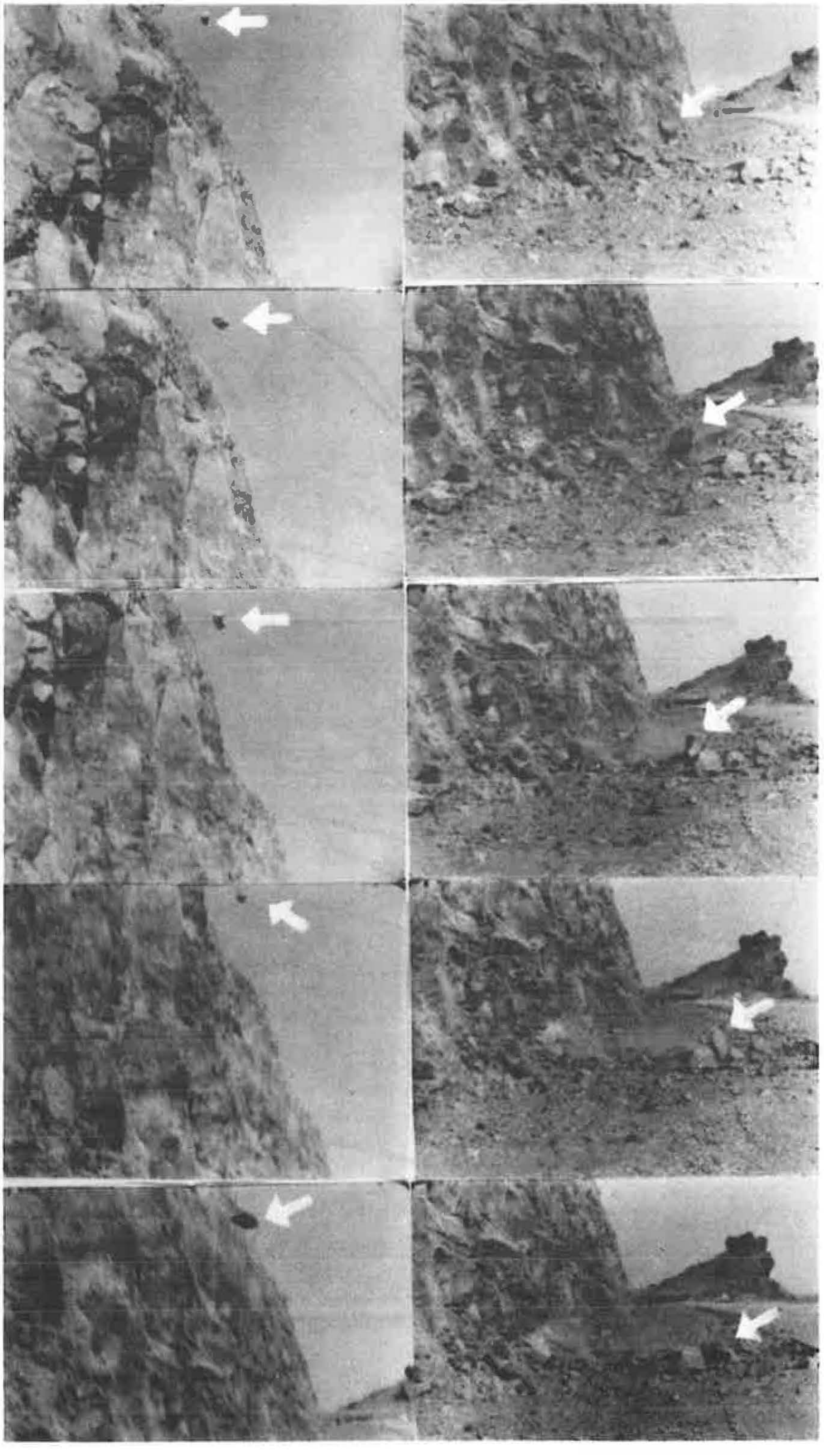


Figure 3. Rock in trajectory, showing that the further it falls, the closer it comes to base of cliff.

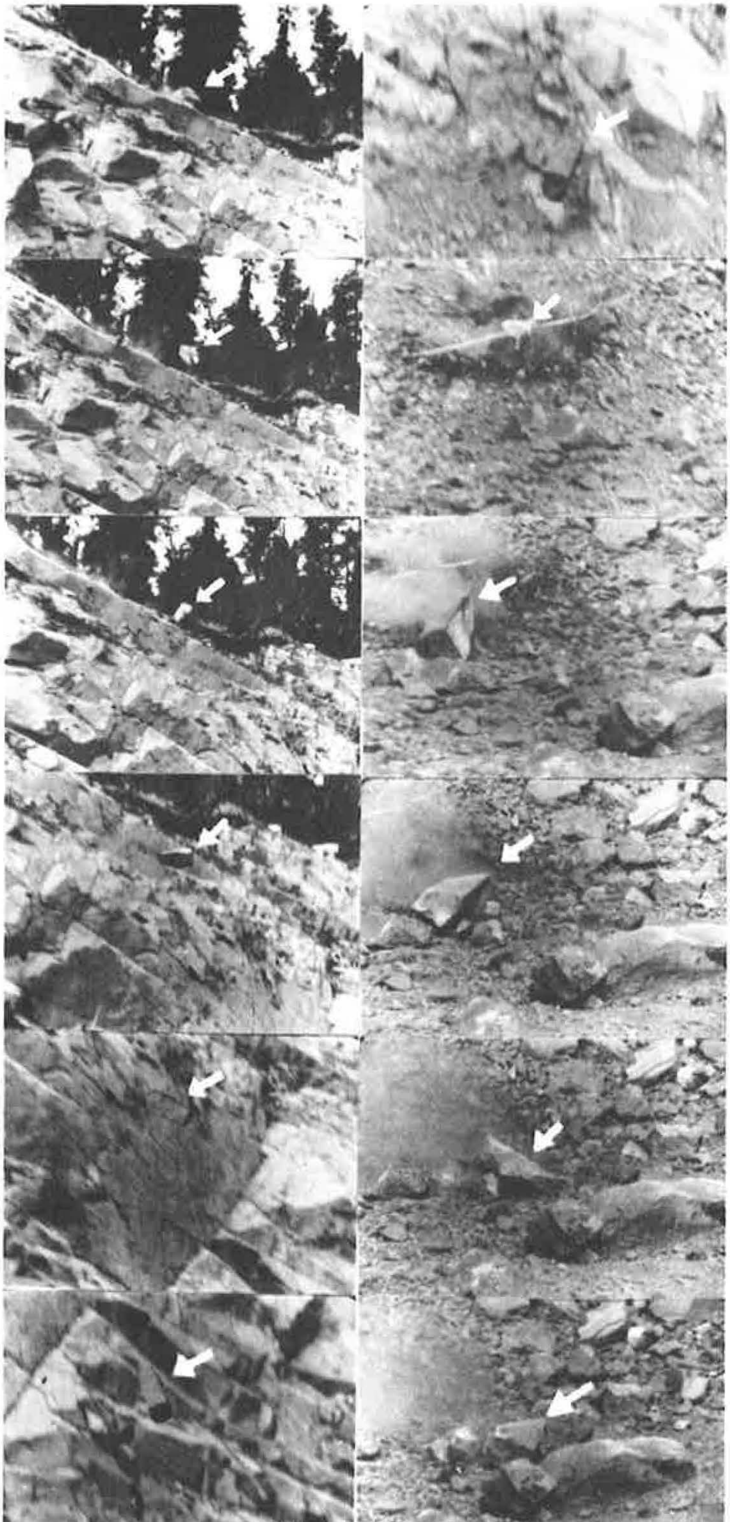


Figure 4. Angular stone with little initial angular rotation rolling after impact (pictures every 10th frame on slow-motion speed of 32 frames per second).

that may suggest what the ultimate strength of a rock mass might be. There is also a need for a means of predicting the stability of the material on the surface of a rock cut. So far, these factors remain elusive and many engineers approach the problem with apathy, as though walking up to a stone wall and half-heartedly demanding that the wall give up its secrets and come under their slide rule.

The unyielding rockfall problem is much like the proverbial wall. A simple method has not been devised for putting together effectively the many factors that govern the stability of rock slopes. At best, an analysis of the existing natural slope is but a clue to the expected internal stability of a rock mass, yet many valley walls are already at the critical angle with active erosion on the surface. A loose rock would seldom stop before reaching the bottom. In this sense, the plane of stability is the surface of the slope, and the problem that faces the engineers is one of determining what the proper slope angle for the proposed rock cut must be, if one is to keep loose rocks from falling onto the highways. The answer is not simple.

Figure 5. Wire netting, rockfall area on old construction.

To illustrate further the complexity of the problem, there is an unweathered granite mass having cubical jointing with its basalt joint system nearly horizontal or slightly tipping into the slope. This, as a rule, produces a combination of factors that tends toward the greatest stability possible. Now, the condition of the degree of weathering of the mass (which does not readily lend itself to a numerical consideration) is introduced. This quality will be especially crucial if the weathering takes place as spheriodal weathering of the joint blocks. In this case, the remaining unweathered round cores of each block have little to do towards strengthening the rock mass as a whole, even though the original joint pattern, which is one of the major controlling factors in stability, is still in the desirable direction. Because of a high degree of weathering, the expression "direction of joint pattern" has been relegated to a factor of little consequence in the stability formula, and is of diminishing value as the degree of weathering of the mass increases. Finally a point is reached where the remaining unweathered cores of each block no longer line up or interlock at all so as to receive support one from another. In this State, the degree of weathering has introduced values of such great uncertainty that there is no clear-cut answer as to the proper slope design. The engineer is required to fall back on educated guess or trial and error.

Over and above such things as degree of weathering, there are still many other physical properties within a rock mass that must also be juggled together to come up with, not the safest but the most practical, stable design. To keep rocks from falling at all would be very desirable, yet it takes only one loose rock for a potential accident. The most practical safe design based on foreknowledge, therefore, has not been constructed to date. Also, the proper slope treatments for any one area may vary widely from those used successfully elsewhere, and both physical and chemical properties of rocks vary widely within States and even more so within national boundaries. Even within the same rock cut not all zones produce the same problem. This condition points up the fact that there cannot be a common formula using the restraining technique, because there are just too many factors involved.

In the North Appalachian and New England areas one can find a great many varieties of rocks, many of which (due to their thickness or lamination, strike, dip, physical and chemical properties, climatic, and other factors) will require different slope treatments. Yet, it is impractical to change slope design within short limits, as would be required.

In some areas, sandstones, limestones, dolomites, and shales are most common. The coastal plains contain crystalline rocks with high mica content. There are massive granite bodies in New England and other areas. Sandstones vary widely in their physical make-up and structural properties. Individual grain sizes vary from fine to coarse, and the bonding agent varies from weak to strong. Some are permeated by numerous hairline cracks. Others are massive and thickly laminated. Some rocks are soft, others hard and brittle. Joints which sometimes are numerous may break in blocks or at acute angles. Alternate layers of hard and soft shales present special problems, especially if they are interspersed with high organic zones. Many of these qualities are within the same rock cut.

Although limestones and dolomites are structurally similar in character, they vary widely in thickness and hardness. Crystalline rocks present special problems, especially where micaceous softer portions separate the denser and thicker seams. Structural qualities of a rock become important—particularly when taken into consideration with road alignment, in which the road may change directions with the prevailing strike and dip of the rock mass, and thus concentrating special unfavorable conditions that may result in landsliding or exceptionally heavy rockfall. In view of these many variables of design, it is easily understood why many areas continue to create a rockfall problem, and that the concept of trying to restrain every rock from falling is faulty. There have been some steps taken in a new direction which use the concept of containing the rocks if they do fall, but this is not going far enough.

Although current designs are taking into consideration fall-out zones, they do not keep the rocks off highways. This shortcoming touches on the related problem of comparative safety, which has inhibited a reasonable solution. If one safety measure comes in conflict with another, it is only reasonable that a subordinate position must be assigned to it for the sake of a new and better design.

As a rule, safety measures do not conflict; but if they do, the preferential one should be that which would cancel out the greatest danger. This point is pertinent in the case of rockfall, because there is a conflict. When gentle off-shoulder slopes, designed for the open road, are brought into rockfall areas, they provide the ramps for stone to come up on the highway. Therefore, there are urgent and compelling reasons to substitute a steep shoulder design in place of the gentle one; for what was thought to be a feature contributing to the overall safety of the highway has now been proven to be, in rockfall areas, a detriment and a hazard.

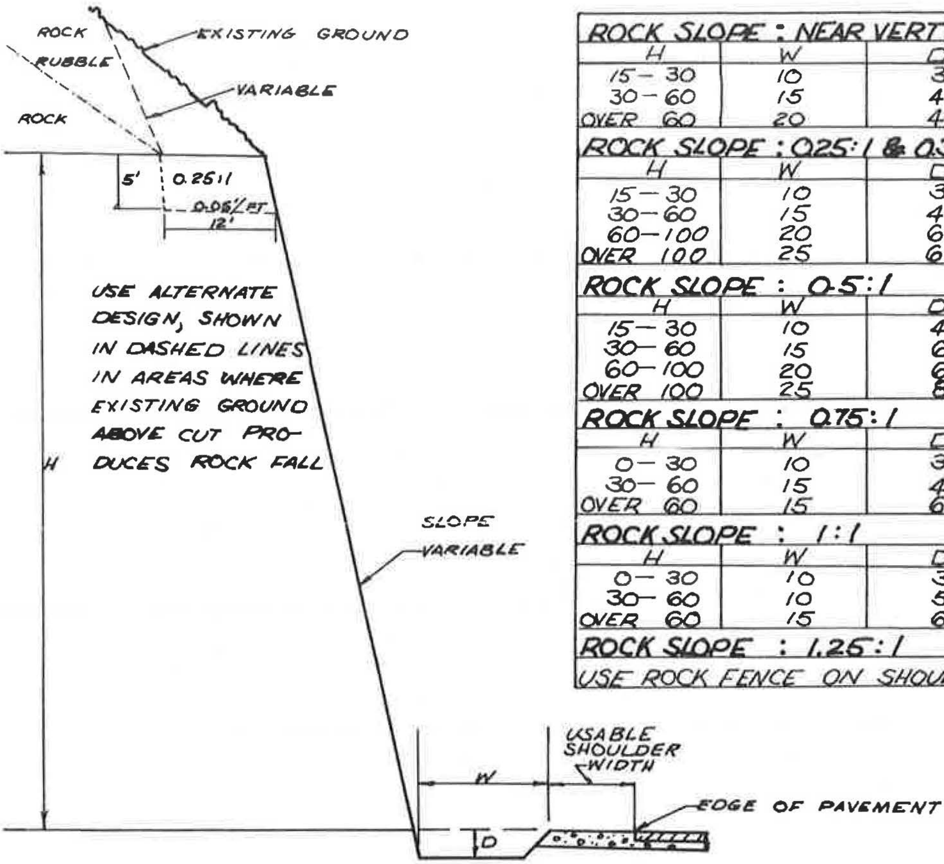
DATA COLLECTION

At the outset it became apparent that a full-scale test was necessary to be able to evaluate properly rock trajectory and other characteristics of rocks in flight. For this reason a 16-mm slow-motion camera was used to make it possible to record hundreds of conditions of rockfall. To keep a pictorial record of these conditions, reference lines (both vertical and horizontal) were employed. It was found that plastic flagging tape was excellent for this purpose, being both colorful and tough. The vertical line was held aloft by small helium-filled weather balloons. The ground end of the tape was anchored to a rock 20 or 25 ft from the base of the cliff. The balloon end was pulled towards the top of the cliff so that the reference tape would be essentially parallel to the face of the cliff. A position of advantage was chosen so that the camera viewed the rockfall from the side to observe its trajectory. Other reference lines were used running parallel along the base of the cliff at 10-ft intervals. It was thus possible for the camera to record the path of trajectory and point of impact by such means. From a study of the impact areas, it was possible to determine the necessary width of the fallout area required for any particular set of conditions. The vertical reference made it possible to determine whether the rocks, at any point, went wider than the proposed fallout zone.

Most of the study was conducted in hard basaltic rocks of all sizes. This proved to be a rock of high resilience and rebound characteristics. Softer rocks, such as sandstones and scoraceous basalts, were much less violent on impact. It is assumed, therefore, that the values in Table 1 giving the relationship between width of fallout vs height and angle of slope will satisfy the maximum demand in most rock work.

TABLE 1

RELATIONSHIP OF VARIABLES IN DITCH DESIGN FOR ROCKFALL AREAS



H	W	D
ROCK SLOPE : NEAR VERTICAL		
15 – 30	10	3
30 – 60	15	4
OVER 60	20	4
ROCK SLOPE : 0.25:1 & 0.3:1		
15 – 30	10	3
30 – 60	15	4
60 – 100	20	6F
OVER 100	25	6F
ROCK SLOPE : 0.5:1		
15 – 30	10	4
30 – 60	15	6
60 – 100	20	6F
OVER 100	25	8F
ROCK SLOPE : 0.75:1		
0 – 30	10	3
30 – 60	15	4
OVER 60	15	6F
ROCK SLOPE : 1:1		
0 – 30	10	3
30 – 60	10	5F
OVER 60	15	6F
ROCK SLOPE : 1.25:1		
USE ROCK FENCE ON SHOULDER		

When required for slope stability the use of benches is satisfactory; however, they do not alter the design and values shown. Ordinarily their use will be a result of the soils study and be on the recommendation of the materials engineer.

Where the existing ground above the top of cut is on a slope approximating that of the cut slope, the height (H) shall include the existing slope or that portion of it that can logically be considered a part of the rock cut.

Ordinarily guardrail shall be provided where D is greater than 3. F permits diminishing D to 4 if fence is also used.

A statewide study was conducted using many quarry faces and conditions. Later, under controlled test procedures, traffic was stopped, and various existing rock cuts were checked. Newly completed rock work also was tested, which had been constructed with fallout zones. In one instance, cliffs from 90 to 130 ft high had fallout zones of 19 and 34 ft in width, and were cut on ¼:1 slopes. Although the excavations were completed with the exception of 4:1 off-shoulder slopes, many rocks rolled as much as 80 or 90 ft from the base of the cliff. Nearly all rocks made their initial impact within a 20-ft fallout zone. It became apparent that if one was to contain the rocks effectively it would be necessary to deflect them back towards the cliff, or provide a wall or buffer condition to stop the roll.

TEST TRAILER

Figure 6. Test trailer used as portable steep shoulder and ditch section (deck = $1\frac{1}{4}$:1 slope).

To implement the conclusion that some device must be placed at the edge of the fallout area to stop rolling rocks, a trailer (24 ft long), heavily-decked with planking and tipped to a $1\frac{1}{4}$:1 slope facing the cliff, was used (Fig. 6). The trailer deck could be set on the ground by removing a few pins and backing into the reach. In effect, the unit became a portable ditch section that was taken from place to place and was used as a deflector or a bumper for rock roll. Whenever the trailer was set at the edge of the fallout area necessary for that type of cliff, no rocks ever went over the top in flight. All rocks were stopped when they rolled up the deck, many of them looping over and falling back towards the cliff. When the trailer was used on flat slopes in the range of 1:1, it proved just as effective in stopping rocks as it did in rock cuts (Fig. 7).

If one replaced the trailer with a constructed slope, it is believed that the natural materials of the slope will enhance the rockfall design, because the rocks will dig into the ground rather than stay on the surface as was apparent with the trailer. One test area was selected which contains much blow sand from the Columbia River in conjunction with some newly constructed rock cuts. It was found that on impact, most rocks lost much of their rebound quality as they dug into the soft sand shoulder slopes. It is not necessary, however, to blanket a ditch area with sand, because fine quarry-run spoils are just as effective.

ROCKFALL FROM TALUS SLOPES

It is difficult to comprehend the magnitude of some talus slopes along mountain highways. It is still more difficult to realize that each of these stones has at some time fallen from the parent cliff and rolled to its present position, only to be buried by other rolling stones. If one lingers long with this concept, the ever-present danger of rolling stones becomes quite apparent (Fig. 8).

Rolling stones are just as dangerous as those generating from rock cuts and it is not uncommon to have talus stones roll completely across the highway, losing themselves over the hill, and obscuring the very existence of a dangerous condition by a mere lack of evidence. Talus slopes are an integral part of rockfall so it becomes necessary to study rock roll in great detail to see if it contains characteristics that might be helpful to the design engineer. Field work consisted of rolling many stones to evaluate their characteristics and to study how the deposits are formed.

The following are some observations that might prove useful: natural talus slopes have curved surfaces concave upward with their flattest portion near the base of the hill. The materials in them are graded imperfectly with the finest materials generally in the steeper portion next to the parent cliff. Each rock in the talus slope has theoretically found its assigned place according to its size, momentum, and slope angle, and conditions of the slope over which it has rolled. Ordinarily, it is the coarser materials that stand at a higher angle of repose; but on talus slopes, the larger materials are found far from the parent source in the flattest portion of the toe region. Obviously, many factors influence the distribution of the deposit; such as angular motion, the sizes of the material on the slope, the momentum of the piece, and subsequent fracture.

If a stone is rolling at high speed along an inclined surface, it touches only the high point of that surface with its own high points. Much of its time is spent in the air and its roll is similar to dice rolling on a rough inclined surface. If the stone is larger than the materials over which it rolls, its angular momentum usually increases until two basic factors begin to diminish its velocity: (a) a flatter portion of the slope, and (b) larger materials over which it must roll. As the rolling stone meets material of

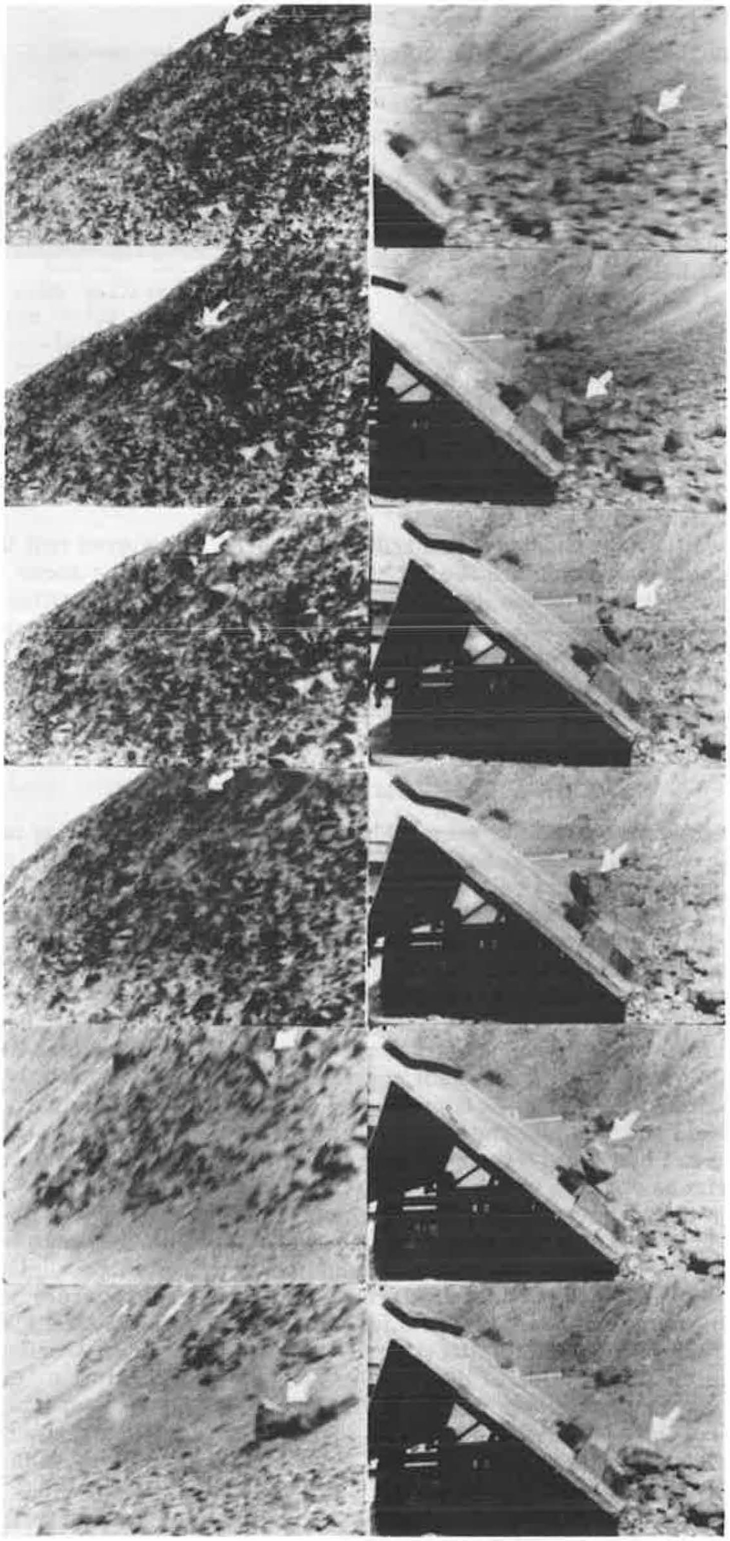


Figure 7. Test trailer used on 1:1 slope of compacted basalt rubble (height of cliff = 110 ft); rock finally rolling backward off trailer deck.

Figure 8. Double talus slope along Columbia River, Wash.; height of cliff in middle background = 400 ft.

its own size, it begins to collide against common pieces, losing energy at first by impact. Progressive slowing-down causes it to fall lower and lower into the irregularities of the surface of the slope so that, as it proceeds, it must raise itself repeatedly over comparable stones. This deceleration process crescendos markedly as the speed of the stone diminishes, so that the stone is soon trapped in voids between stones of its own size, resulting in a segregation of the materials. If for some reason a large stone gets stopped on the slope higher than normal, it may cause a deposition of similar material around it which would not ordinarily have stopped there (Fig. 9).

Continuous rockfall causes talus slopes to grow so rapidly that original conditions will change at any one elevation. Talus slopes, therefore, become stratified and closely resemble the cross-section of a river delta. Once a stone is trapped on the surface, it never, of itself, begins to roll again (although much of the material may move about by debris sliding). On mature natural slopes, rock roll is seldom a problem for highways located near their base; but an excavated slope is not a talus slope because it now stands at the angle of repose of the material. In many cases, cut slopes or backslopes may reach hundreds of feet above the roadbed and a stone once started on this slope cannot stop until it reaches the roadway below. Figure 10 shows a talus slope cut just steeper than the bedding plane of the deposit. This causes popouts and progressive failure of the slope. In cases where long backslopes in talus material exist, some protection for the traveling public is most desirable, if not mandatory.

If a roadway location is placed high on the slope it generally requires a greater amount of material to be handled than one lower down for any given roadway width. This is due to the curvature of the talus surface and the flatter angle of repose of the finer materials near the top of the talus pile. Of course, this is true only for locations that do not intersect the cliff face above. Generally, there is a critical maximum for a roadway width. Any additional widening beyond this critical point, such as the construction of wide ditches as a means of containing rock roll, becomes economically prohibitive. For this reason, a special rock fence was tested and is offered as a device to arrest rock roll in lieu of a wide ditch area as a catch basin feature.

ROCK FENCE

The rock fence is designed to decelerate a rolling stone and to retard its angular velocity. Nearly all rocks stay very close to the talus surface while they roll. This characteristic permitted a chain link fence only 6 ft high to encounter all rolling stones. The fence is novel in that it is not mounted like an ordinary fence, but is suspended like a curtain from a cable. The cable, in turn, pulls on a compressive spring to absorb the shocks of the rocks. The spring that was used for the test was one taken from an old cable guardrail set-up. The cable is supported by fence posts every 50 ft to keep the top of the rock fence essentially in a horizontal line. The installation is mounted on the slope above the ditch line, or on the back side of guardrail posts on the shoulder of the road (Fig. 11).

SOME OBSERVATIONS ABOUT ROCKFALL

There are many factors to analyze in considering the characteristics of rock trajectory. Some of these are size and shape of the stone involved, angle of slope, surface characteristics of the slope, height of cliffs, broken slope features, kind of rock, gravity, and time. Common to all rockfall, however, is the factor that rocks start

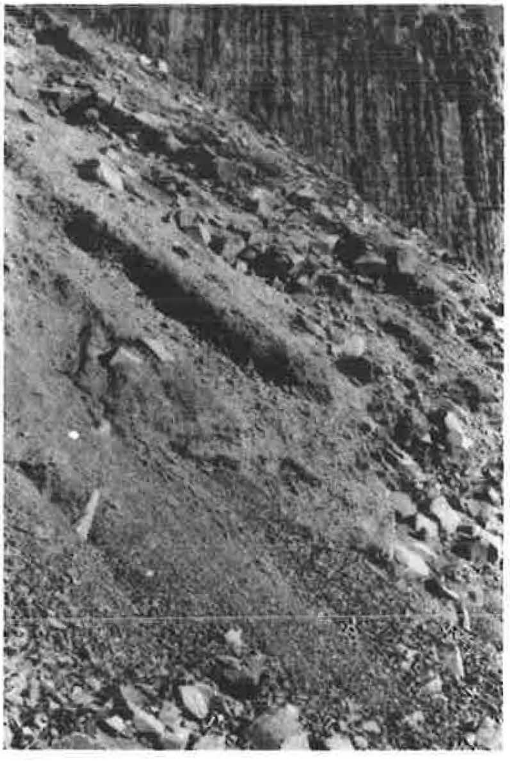


Figure 9. Partial cross-section through talus material, showing fine grading of materials under surface layer of boulders.

Figure 10. Bouldered layer on surface in Fig. 9 stripped off, showing segregated strata deposited flatter than angle of cut slope.

from a static position by rolling and then begin their descent. Depending on conditions, they will either continue to roll or take one or two bounces and go into flight (Fig. 12).

Rocks can arrive on the roadway after one or more of the following modes of travel: sliding, rolling, skipping, and vertical fall. Recent field studies have caused engineering assumptions regarding safety requirements to come into question. It is conceded that several simplifications are necessary to keep analytical considerations manageable, but it is felt that results agree rather well with empirical data advanced.

A spherical rock mass is assumed to be moving under the following circumstances.

Case 1. A smooth, uniform, inclined slope is assumed. (Experience shows that most rocks "stay put" on the slope up to an inclination of about $1\frac{1}{3}$:1. Steeper slopes cause the rocks to begin to roll at an accelerated rate.) If the stone and the plane are perfectly smooth, then the rock will roll continuously on that surface, regardless of the inclination of the plane. Although this condition is never reached fully in nature, it is approached when the slope is composed of fine materials.

Case 2. A broken slope or a rough slope producing "ski jump" features are added to Case 1. This interesting phenomenon deals with impact. Impulsive forces act for a very short time, yet are of great magnitude. When there is impact of a falling particle against another body (such as a rock on a cliff face), there is a change of velocity that depends on the nature of the materials and angle of impact. (It is observed in nature that rocks originating on steep slopes first begin their descent by rolling, then, usually after one or two short bounces, go into trajectory. Seldom do these rocks touch the slope again within the practical limits of ordinary rock work. This is true especially for slopes that are $\frac{1}{3}$:1 and steeper.)

In view of the fact that some quality of both cases cited will always exist, theoretical

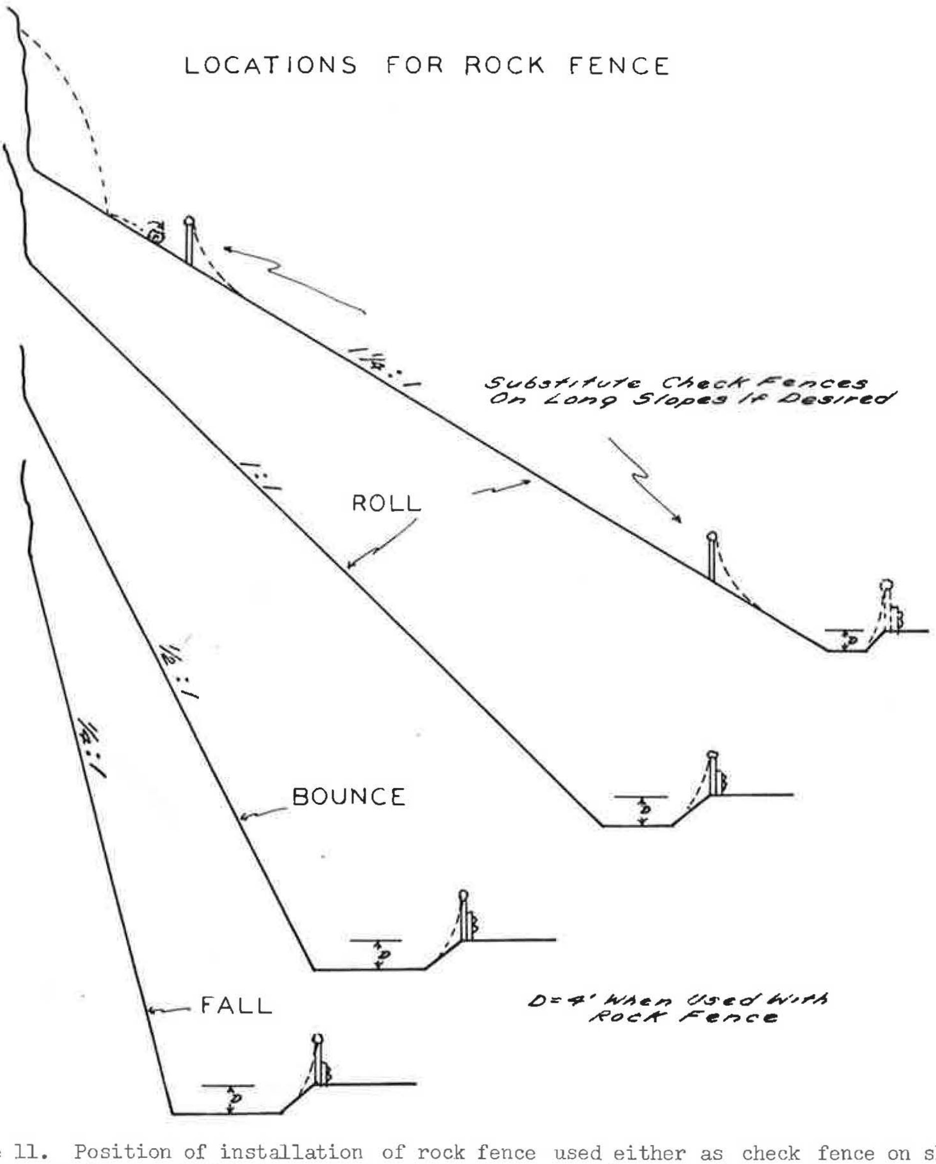


Figure 11. Position of installation of rock fence used either as check fence on slope or as protection on highway. Though ditch design for $\frac{1}{4}$:1 and $\frac{1}{2}$:1 for a 100-ft high slope is same, yet rock from $\frac{1}{2}$:1 slope must cover a greater distance before encountering same fence.

physics can offer a description of the motion of the falling rock. For example, a sphere is in trajectory with mass m, angular velocity a_o, translation velocity v_o, in trajectory at angle θ_o with the horizontal, impinging on a plane A-B that makes an angle of θ' with the horizontal (θ_o and $\theta' < 90^0$). To simplify the algebra, an origin is placed at time t_o when the sphere first comes under consideration. The X-axis is horizontal and the positive direction of the Y-axis is downward. The trajectory may be described parametically for any time t and for any inclination of plane A-B.

$$\text{(parabola)} \begin{cases} x = v_o (\cos \theta_o)\ t \\ y = v_o (\sin \theta_o)\ t + \frac{1}{2} g t^2 \end{cases}$$

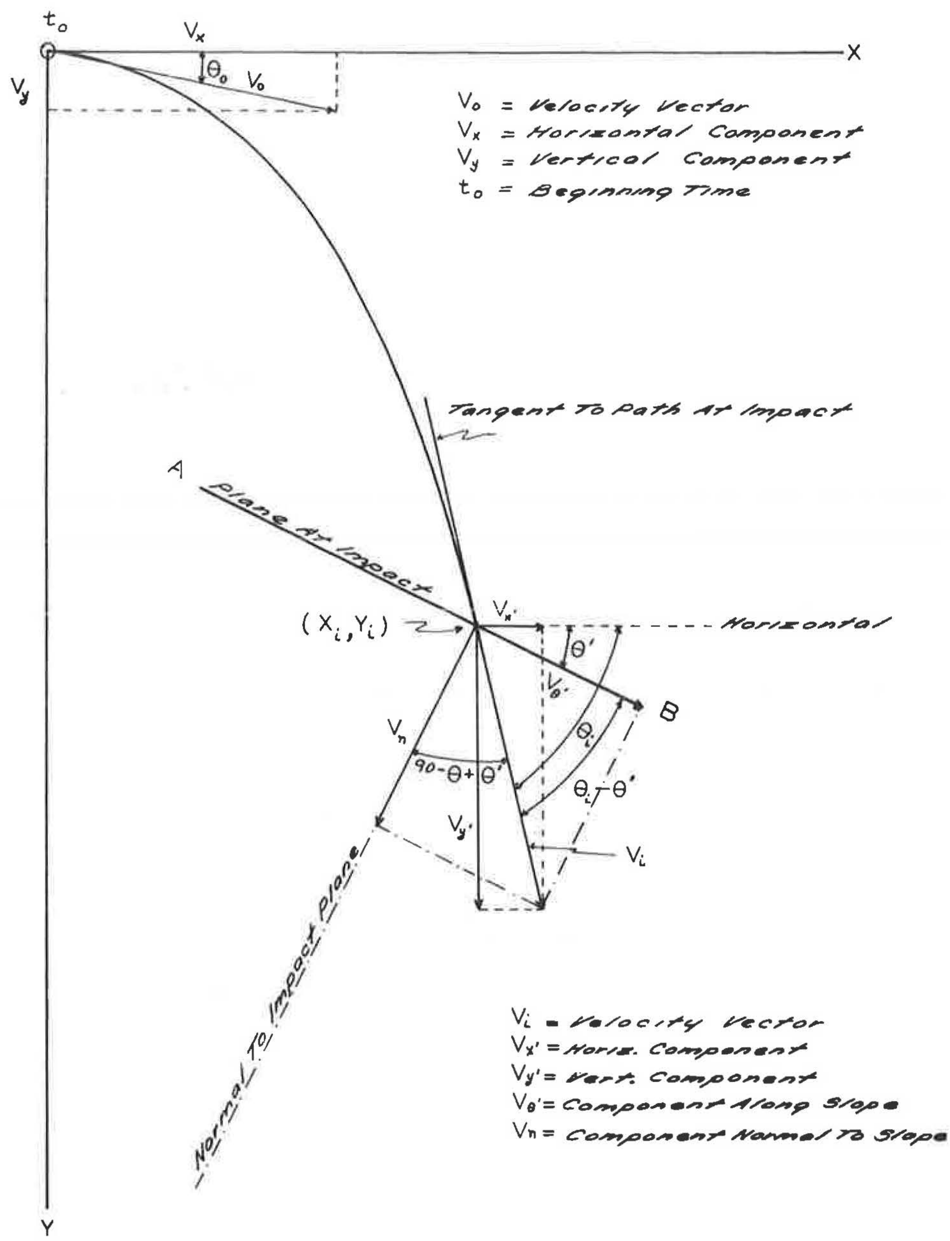


Figure 12. Theoretical consideration of rock trajectory.

in which g = acceleration due to gravity.

If (x_i, y_i) is the point of impact on A-B, and it is assumed that $a_o = a_i$ and a_i = angular velocity at t_i,

$$v_i = \sqrt{v^2_{x'} + v^2_{y'}}$$

$$\text{(constant)} \quad v_{x'} = v_o \cos \theta_o$$

$$v_{y'} = v_o \sin \theta_o + gt_i$$

$$v_i = \sqrt{(v_o \cos \theta_o)^2 + (v_o \sin \theta_o + gt_i)^2}$$

$v_{x'}$, $v_{y'}$ are the horizontal and vertical components of v_i.

On impact, two major actions occur: First, the sphere rolls down the slope due to both its angular velocity and gravity. Coefficient of friction and stability of surface are factors of considerable importance on talus slopes but are of less importance in rock cuts. Second, compression and restitution phases of impact based on elasticities tend to send the sphere into trajectory. The worst condition possible (greatest horizontal impulse) is given when the vector of v_i (velocity at impact) makes an angle of about 45^0 with the plane A-B. In either situation, the force normal to the impinged plane should be determined. If θ_i is the angle with the horizontal that the sphere makes at t_i, then

$$\tan \theta_i = v_{y'}/v_{x'}$$

$\theta_i - \theta'$ = the angle between the trajectors at t_i and the plane of impact; $90^0 - \theta_i + \theta'$ = the angle between the trajectory and the normal to the plane; $V_{\theta'}$, the component along the slope, will accelerate the roll during contact; V_n, the component normal to the surface, will be a factor in the rebound from the surface.

If the impinging mass is not spherical an appreciable movement will be imparted, causing a change in angular velocity.

Angular momentum is a very pertinent part of the rockfall study and it is not only involved in rock roll on cliffs and on talus slopes, but is one way by which the inertia from a rock in trajectory is dissipated and transferred on impact. It was observed that even a falling stone revolving backward from the direction of normal roll would, after impact, pick up angular momentum in the opposite direction. Further, if a falling stone had considerable angular velocity to start with, it could be immensely increased after impact.

In a general way, rocks that roll on ½:1 slopes are given the greatest horizontal impulse, and are the most dangerous as far as controlling them is concerned. In the rare cases where a rock may strike the face of a steep cliff the second time, the piece is traveling at such high velocity downward that even though it rebounds again, it does not have time to cover much horizontal distance; hence, it still makes impact within the fallout zone. Such stones have tremendous horizontal momentum, however, and roll considerable distances after impact. If a rock falls from a vertical cliff, it usually has little or no angular momentum or horizontal impulse to start with; thereupon, after making impact at right angles to the ground, it calmly comes to a stop. In considering the kind of backslope to design, the height of a cliff is very important. A rock falling from a 40-ft high cliff on a ¼:1 slope may roll further from the base than a rock starting from the same height on a ½:1 slope. This is due to the fact that there is not sufficient time in the latter case for a rock to gain appreciable momentum, whereas the falling rock picks up momentum quickly. But with higher slopes, in the range of 100 ft, the ¼:1 slopes permit a rock to make nearly vertical impact, whereas ½:1 slopes induce a rock to have tremendous horizontal momentum and angular velocity.

Size and shape of a rock have little bearing on its falling or rolling characteristics. The greatest difference is with large rocks that stay close to the face of the cliff and may from time to time touch on the way down. The shape of a rock has little importance unless it is long, like a pencil, which retards roll and gives eccentric action. Flat or angular-shaped rocks made little difference. Perhaps this is analogous to dropping a handful of pennies and dice on the floor and noting the distances that they roll from point of impact. Rocks that fall in trajectory seldom give a high bounce after impact but rather, change their linear momentum into angular momentum. It is this feature that permits a steep off-shoulder slope or a steep slope with a fence to contain them.

SUMMARY

Because a falling rock must obey certain natural laws of mass, energy, velocity, impact and restitution, as well as being influenced by friction, time, gravity, and other things that have been rather well-known since Newton's time, it seems only reasonable that the behavior of falling stones would lie between certain limits, limits that can be used as a basis for design to contain them.

Field work has supported this concept, permitting the use of (a) fallout areas in which to dissipate the enormous energy arriving at impact; (b) steep off-shoulder slopes to combat the angular momentum of the rock generated after impact; (c) rock fences as a flexible buttress and decelerating device to dampen off angular velocity and to smother the rock if it becomes necessary to contain it.

A new approach to the rockfall problem has been opened. Old concepts have been evaluated in the light of new data and must be discarded along with other obsolete ideas, such as "the larger the rock, the faster it falls."

Practical value has resulted from this study in that untold savings will result from new design, and a generous share of practical safety has been served to the traveling public.

ACKNOWLEDGMENT

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The Peace River Highway Bridge— A Failure in Soft Shales

R. M. HARDY, Research Professor of Civil Engineering, University of Alberta, Edmonton

This paper deals with the failure of the Peace River highway bridge and the stability of slopes in soft shales formed by heavy overconsolidation of alluvially deposited clays under the weight of glaciers. Geologically these are classed as soft rocks, but in many instances no cementation of the particles has occurred. The only physical alteration to the original clays has been compression due to the consolidating pressure from glaciers during the Pleistocene period.

These shales are presently in a state of rebound and on release of overburden pressure, along with access to water, they may revert to soft clays, frequently having high swelling characteristics. The mineralogical constitution of the clay minerals does not appear to be the predominant factor governing the swelling characteristics. Rather, these appear to vary with the physical-chemical environment of the pore water and adsorbed moisture films.

Such soils are of wide occurrence in the northwestern portion of the North American continent. The paper deals with the performance of such materials in engineering construction in northwestern Canada, and points out the deficiencies of conventional methods of analysis for predicting their behavior. The swelling mechanism and its significance are discussed, and suggested improvements in concepts for analyzing the stability of slopes in such materials are presented.

• OVER EXTENSIVE AREAS of the North American continent lying in a strip several hundred miles wide to the east of the Rocky Mountains there occur shales in which the major diagenetic process of formation has been overburden pressure. These shales now exist at overburden pressures much reduced from the maximum that has occurred in their geological history. They are in a state of rebound and under certain environments tend to revert to clays. They are frequently described as "clay shales."

This paper deals with engineering experience, particularly in regard to slope stability in highway construction and bridge approaches, with such clay shales occurring in northern Alberta, northeastern British Columbia, and the western portion of the Yukon Territory in Canada. They are of Cretaceous age and occur interbedded with coal seams, silt, sand, siltstones and sandstones having a wide variation in quality of cementing media. Many of them have an appreciable organic content. It has been estimated that they have been subjected to the weight of at least 1,500 ft of sediments and ice in addition to their present overburden, during the various periods of glaciation since their original deposition. They now frequently lie under a shallow overburden of recent deposits. The clay shales extend to depths of hundreds of feet, and the major rivers in the area are incised to depths of as much as 800 ft in these deposits.

Several cases of spectacular slope failures have occurred for which extensive soil investigations were subsequently performed, and for which fairly comprehensive data

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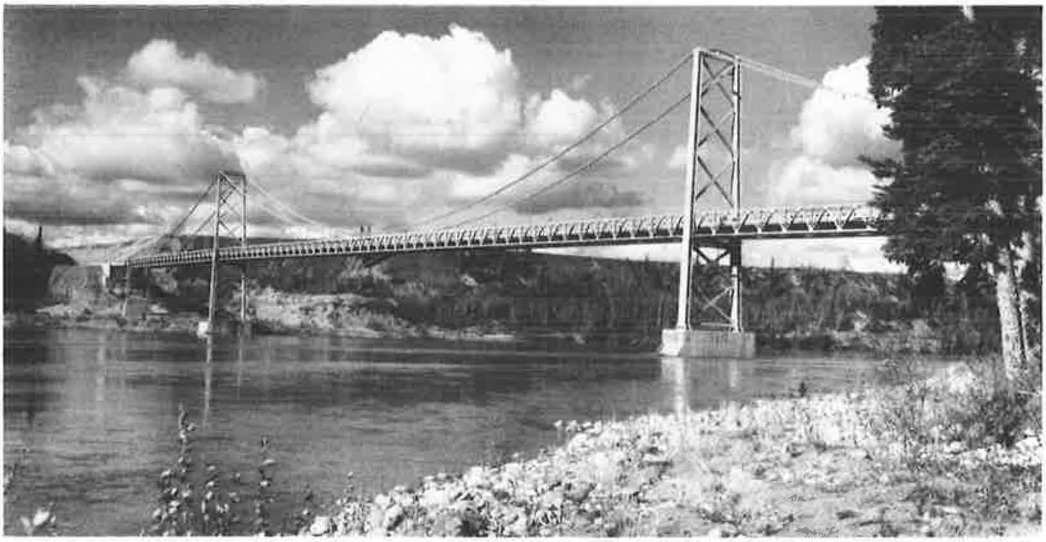

Figure 1. The Peace River highway bridge as originally constructed.

concerning the subsoil conditions have become available. This paper deals specifically with one of these failures which involved a major bridge collapse and which has not previously been reported in detail in terms of these data. A comparison is also made with the results of two other case histories involving slides in these soil types.

The particular case described in detail is a slide which developed at the north end of the Peace River highway bridge at Taylor, British Columbia. It resulted in the collapse of the bridge on October 16, 1957. The history of the performance of this bridge is given in considerable detail because it is significant to an understanding of the causes of the slide and has not previously been comprehensively reported.

Figure 1 shows the bridge as originally built, looking towards the north bank on which the slide subsequently developed. It was built in 1942 incidental to the construction of the Alaska Highway, now called the Northwest Highway System in Canada, and was designed by the United States Public Roads Administration. It was a suspension bridge with a total length of 2,130 ft, made up of a main span of 930 ft, suspended side spans of 465 ft, and 135-ft simple trusses spanning from the cable bents to the anchor blocks at each end. At the north end the tower and cable bent were carried on spread footings on shale, and the anchor block was also based on shale.

The topography at the north end of the bridge was characterized by a gravel terrace flat about two miles wide at an elevation about 200 ft above the river level. The original slope of the bank at the edge of this flat was close to 1.5:1 down about 170 ft to a second flat about 200 ft wide and about 30 ft above the river level. At the time of construction the main river channel was close to the south shoreline, but by 1949 it had shifted so it was encroaching along the north bank.

The geology in the area of the north bank is characterized by a gravel terrace on the upper flat overlying shale at a depth of about 100 ft, with the contact surface being irregular. The shale has been described geologically as bedrock, and is a black to gray marine shale of Lower Cretaceous origin. The formation is several thousand feet thick in the area. The shale is soft and thinly bedded and frequently includes interbeds of silt. The overlying gravel on the terrace is coarse cohesionless sand and gravel with a slight silt content and is in a medium dense state.

The natural moisture contents ranged from 3 to 35 percent. The lower moisture contents occurred at depths below the surface generally exceeding 50 ft, and for intact shale, which had not been subjected to disturbance or weathering, the moisture contents were generally less than 10 percent. The plasticity values for the shale were all within the medium-to-low plasticity range. They plotted parallel to and slightly above the "A line" on the Casagrande plasticity chart. Laboratory unconfined compressive strengths ranged from about 0.5 to 20 tons per square foot and averaged about 10 tons per square foot. One test gave a strength of 32 and another 56 tons per square foot. Consolidation test results were all characteristic of heavily over-consolidated soils. By extrapolation beyond the preconsolidation load, a compressive index value of about 0.26 was indicated. In general the samples did not indicate high swelling characteristics in the consolidation test, but a maximum swelling pressure of 5.0 tons per square foot was recorded in a "free swell" test with the sample immersed in distilled water.

At the time the bridge was located the site gave the general appearance of being one of the most stable possible locations in the area, except possibly for the fact that a series of springs existed at the contact of the gravel and the shale. However, two things incidental to the bridge construction are particularly pertinent to its subsequent behavior. One of these was that a cut to a depth of about 50 ft was made in the gravel terrace to secure a satisfactory grade for the approach road to the bridge. This removed an appreciable overburden weight from the shale in the area of the north anchor block. The second unfavorable factor was that surface runoff for about a mile north of the bridge was carried down the road ditches and discharged on either side of the bridge anchor block. This drainage area was further increased in 1955. Thus, surface runoff was made available to penetrate through fissures in the shale in the area of the anchor block, a condition that did not exist prior to the bridge construction.

In 1947 it was found that scour had occurred under the foundation pad for the north tower, despite the fact that at the time of construction the foundation excavation had to be taken out with jackhammers and it was considered to be rock excavation. The tower foundation was underpinned in 1948. Some additional scour occurred around the north tower foundation in 1952 and this was corrected by riprapping.

In 1952 there was visual evidence that the bridge deck had lost some of its camber and there appeared to be the possibility that the north anchor block had moved horizontally about 3 in. At that time the Canadian Army, which operates the Northwest Highway System, established a series of check points on the bridge foundations and deck. During the summer of 1957, a year of higher than average precipitation in the area, severe scour occurred on each side of the north anchor block due to high intensity of surface runoff. In September of that year there was visual evidence of possible movement of the north anchor block.

In view of this, officials of the Northwest Highway System made a complete check of their 1952 survey during the period October 3-8, 1957. This showed a possible 1.08-ft shift southward and a definite 3-in. shift westward of the top of the north cable bent. Observations on the north anchor block showed a tilting, as well as shifting toward the south, with a possible movement of 1.6 ft horizontally in the interval 1954 to early October 1957. It is likely that most of this movement occurred during the late summer of 1957, because with this magnitude of total creep noticeable defects in the approach road immediately north of the anchor block would be inevitable. Such in fact were evident between early September and early October 1957. A total settlement estimated at 18 in. in the approach road was observed for a distance of about 18 ft north of the anchor block during this period, cracks were observed in the soil on either side of the anchor block, and surface runoff disappeared behind the anchor.

During 1956 and 1957 a natural gas scrubbing plant was erected on the gravel terrace a few hundred feet to the north and east of the north end of the highway bridge. This involved the erection of a water intake plant at the river with supply and return water lines to and from the plant to handle up to 18,000 gpm. The water intake structure was located about 650 ft west (upstream) of the highway bridge, but the pipelines to and from the plant were carried along the lower bench, below the highway bridge and up the bank about 200 ft east of the centerline of the bridge. The pipelines were constructed with a minimum of disturbance to the natural soil conditions along the bench and up the bank. Their construction involved leveling the surface along the bench, but fill was placed over them rather than placing them in an excavated trench. However, they were placed in an excavated trench up the face of the bank to the east of the bridge.

Construction of the intake structure was commenced in the early winter of 1956-7 at a site about 150 ft east of its final location. It required about a 50-ft deep excavation on

Figure 2. Gas processing plant and pipeline suspension bridge in relation to highway bridge.

the lower bench. Freezing of the walls of the excavation was intended to provide stability during construction. However, heavy freezing temperatures did not occur until early January, and a few days previous to this the north wall of the excavation caved. The structure was then moved 150 ft to the west and founded on steel H-piles driven to refusal in the shale. Some of the piles were battered. This proved to be a fortuitous occurrence in view of the subsequent bank movement.

A natural gas pipeline suspension bridge across the river was also erected during 1956 and 1957 at a location about 700 ft east (downstream) from the highway bridge. This was built with its tower foundations and wind cable anchors on the lower bench and its main cable anchor block in the gravel on the upper bench. These foundations were all built with as little disturbance as possible to the natural soil conditions.

The locations of the natural gas scrubbing plant pumphouse and pipeline suspension bridge in relation to the highway bridge are shown in Figure 2.

The gas processing plant was put into operation during the summer of 1957. No difficulties were encountered with the water supply until October 15th.

Precipitation records in the area have only been available since 1928. An analysis showed that the period June 1 to October 15, 1957, was the wettest on record, and that statistically its occurrence would be at a frequency of once in 30 years. It is also significant that the average precipitation during the annual periods May 1 to October 15 from the time the bridge was built in 1943 until 1956 was 10.2 in., whereas the precipitation for this period in 1957 was 18.0 in. Moreover, in September and early October 1957, there were two heavy falls of wet snow, which would result in a greater percentage of the water going underground into the gravel than would usually occur with rainfall.

At 1:00 p.m. on the afternoon of October 15, 1957, and before officials of the Northwest Highway System were able to arrange for specialist advice following their survey of the conditions in the period October 3-8, the gas processing plant operators detected that the water supply system was not functioning properly. By 3:00 p.m. rocks and soil were being discharged in the return line, and water was flowing from the bank part way up the pipe trench. By midnight that day it was evident that all the water lines were broken. However, attempts were made during the afternoon to keep the plant in operation so that water under pressure was in the lines until about 3:00 a.m. the following morning.

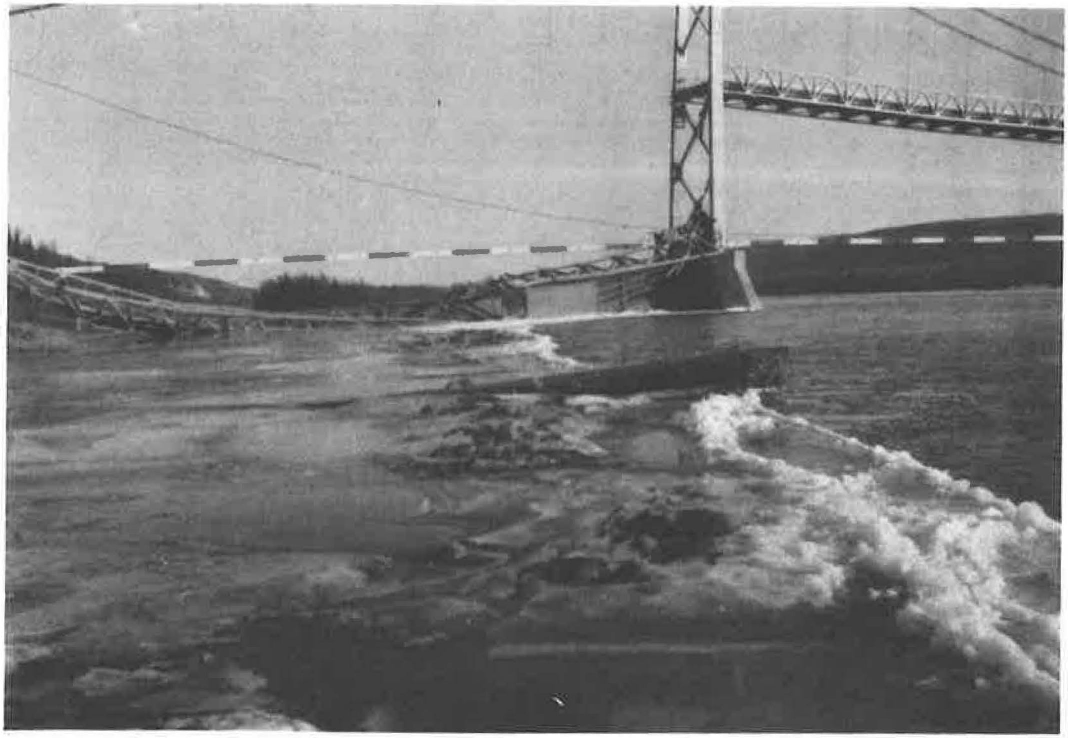

Figure 3. Toe of slide projecting above river surface.

No evidence of additional movement around the north anchor of the bridge was observed until 2:15 a.m. on October 16; at this time a depression of a few inches had occurred in the road surface at the north end of the anchor block. This depression slowly increased, and it was necessary to close the bridge between 4:00 and 5:00 a.m., and by morning the slide was well developed. Collapse of the two north spans of the bridge occurred at 12:40 p.m. on October 16.

It was evident following the collapse that the bank instability was not confined to the area immediately around the north anchor block. It was clear that a major bank movement had developed. The slide extended a distance of about 470 ft east of the highway bridge, and about 625 ft west to within a few feet of the water intake structure. It was thus about 1,100 ft long and was centered about 75 ft west of the bridge centerline. The extent of the slide above the river level was clearly defined by a sheared face at the top and cracks at the ends. The toe of the slide appeared above the water surface at a distance of about 120 ft from the shoreline.

The extent of the bridge collapse is shown in Figures 3, 4, 5 and 6. The toe of the slide is shown in Figure 3 projecting above the water surface in the river. The extent of the slide which developed in the bank is indicated in Figures 6 and 7.

Neither the water intake building nor the pipeline suspension bridge were moved by the slide, except that a wing wall at the east end of the pumphouse was undercut by the movement. However, the water lines were completely wrecked within the slide area, and the distortion of these from the bank movement caused some damage within the pumphouse. These damages, however, were negligible as compared to the damage to the highway bridge. It was subsequently dismantled and replaced by a new bridge.

The cause of the bridge collapse was without question the instability of the bank. The weight of soil within the mass which moved far exceeds the forces acting on the bridge anchor block. It was not a structural failure of the bridge in any sense. The anchor block and cable bent foundation merely rode the slide. However, it appears that deteri-

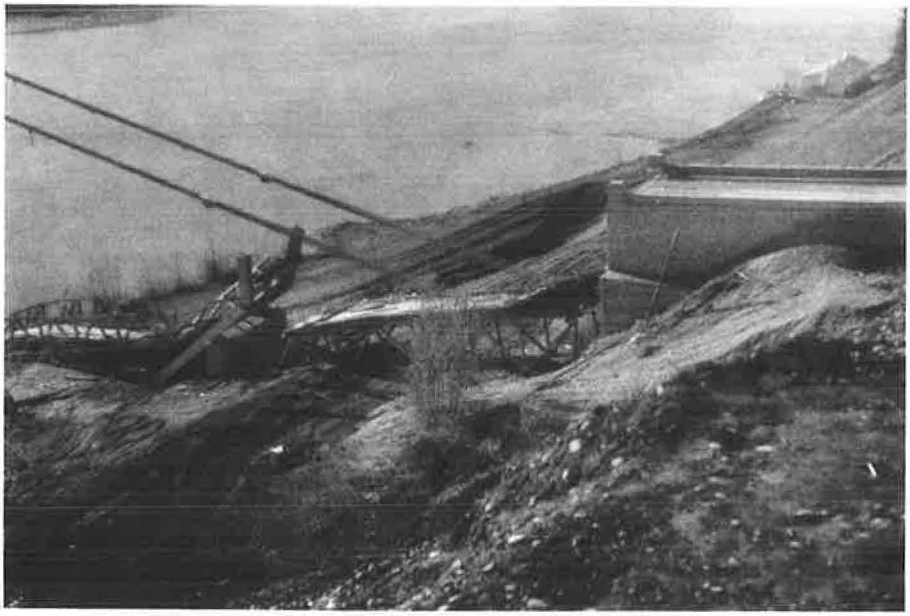

Figure 4. Slide area, showing movement of anchor block.

oration of the stability conditions in the bank commenced soon after completion of the bridge and they continued to deteriorate for a period of years.

The sequence of events and the known characteristics of the shales existing in the bank suggest that the approach road cut into the natural bank at the north end of the bridge, plus the concentration of surface runoff in the highway ditches on either side of the anchor block, were factors that probably initiated the deterioration of the shale. The excessive rainfall during the previous summer undoubtedly accelerated the deterioration of the subsoil conditions. However, the intensity of rainfall in that period was not greater than should have been normally expected throughout the life of the bridge.

Of more problematical significance is the effect of the construction activities in the area of the slide during 1957, particularly the construction of the water lines. There is no evidence in the sequence of events leading up to the collapse that the construction activities had any appreciable effect on the stability conditions. However, it does appear that the broken water lines evident for the 24-hr period previous to the collapse greatly aggravated the final deterioration of the stability conditions. If the broken water lines had not been in existence, the rate at which the conditions progressed towards collapse would undoubtedly have been much slower, and possibly would have been such that remedial measures could have been taken before the bridge was finally destroyed.

It is, of course, pertinent to inquire as to whether the water lines broke due to defects inherent in one or another of them, or whether the creep of the bank broke a water line and then the loss of water into the bank created seepage pressures and accelerated the stability deterioration. This question can be answered only by inference. The lines were all pressure-tested before the plant was put into service, and the failure of pressure-tested pipes due to inherent defects is rare in pipeline experience. In view of the history of creep of the bridge anchor over the years, it is at least logical to conclude that it was movements in the bank which ruptured the water lines.

Extensive investigations were made by several authorities following the collapse to establish the reasons for the failure and to assess the stability of the water intake and pipeline bridge structures located adjacent to each end of the slide. Efforts were, of course, made to identify the position of the failure surface. This proved difficult to positively identify. However, in two test holes significant increases in natural mois-

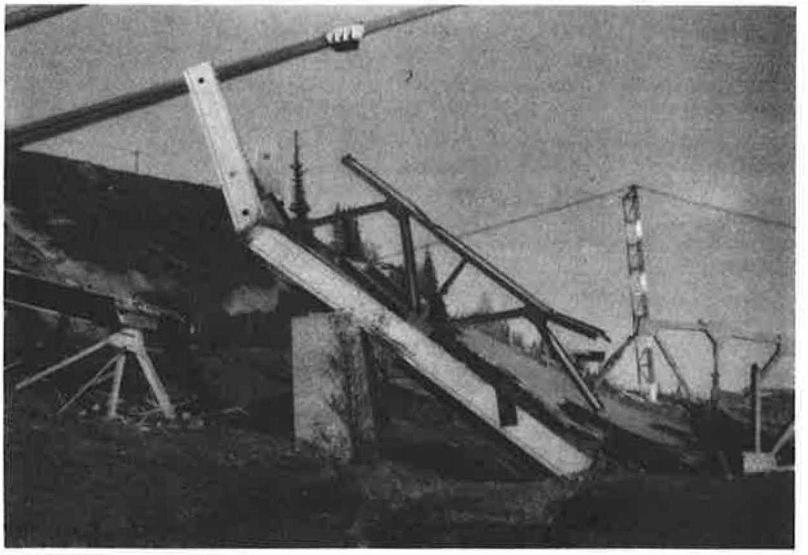

Figure 5. Collapsed cable bent.

ture contents occurred and some evidence of soil disturbance was detected at elevations 1285 and 1290. These were 30 and 35 ft, respectively, below the river level at the time of the collapse. Although the natural moisture contents in this zone showed a bulge in the moisture content profile for depth, they were only about 20 percent.

All samples taken were badly fissured and difficult to handle for test purposes. However, one series of carefully run consolidated undrained triaxial compression tests with pore pressure measurements defined an effective angle of internal friction of 30.6^0 with a cohesion value of 0.75 tons per square foot. However, the data could have been interpreted equally as well to give an angle of internal friction of 36^0 with no cohesion.

Stability analyses were made for various cross-sections through the slide area. On the section along the centerline of the bridge with the forces of the anchor block acting, a sliding block analysis with its base at elevation 1285 at the toe and 1290 below the anchor block gave the most realistic results. Using an effective stress analysis with an angle of internal friction of 30.6^0, an excess hydrostatic head on the base of the sliding block closely equal to the elevation of the contact between the shale and the gravel of the terrace was required for a factor of safety of one.

This is a realistic result to the extent that the broken water lines could be expected to produce a head in the area of the anchor block that would be close to the contact between the pervious gravel and the underlying clay shale. The seepage pressures would be transmitted through the shale to the failure surface through fissures. However, it is unrealistic in several respects. First, the cohesion component of shearing strength has been neglected in the analysis. Second, if the alternative interpretation of the soil test data is used by which the cohesion is zero and the angle of internal friction is 36^0, a considerably greater hydrostatic pressure is required on the failure surface for a factor of safety of unity. Third, the analysis has been made assuming the forces from the anchor block were resisted entirely by a slice of soil through the slide of width only equal to the width of the anchor block; if they had been distributed over the 1,100-ft width of the slide area, the average soil strength required to resist them would be much less. Fourth, failure conditions were imminent before water from the broken mains was available to provide the high seepage pressures on the failure surface necessary to show a factor of safety of unity. The effect of each of these factors is to lower the actual average shearing stress existing on the failure surface as compared to what can be estimated as being available using conventional concepts of the shearing strength of soils.

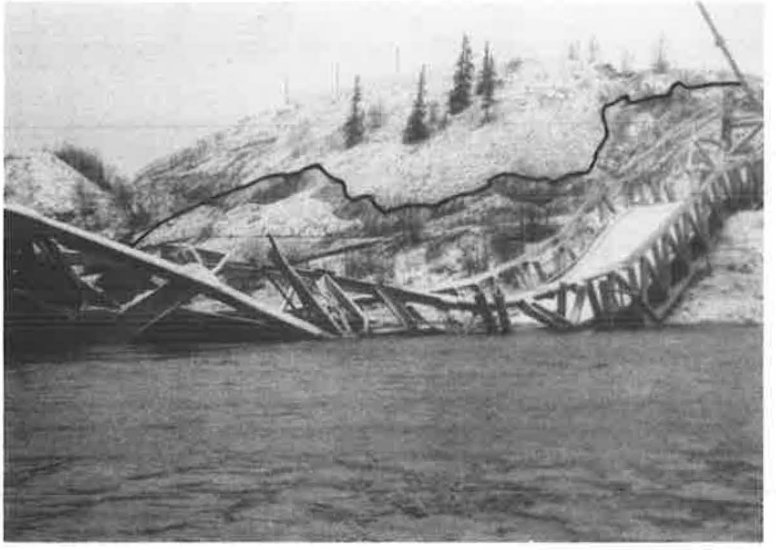

Figure 6. Slide escarpment to west of highway bridge.

Figure 7. Slide escarpment to east of highway bridge.

Within the scope of this symposium this case history illustrates a situation in which, at the time of the original construction, the soils involved at the site were considered to be bedrock and they had the properties of soft rocks. However, normal construction practices appear to have had the result of greatly accelerating the disintegration of these rocks. The result has been that within a period of only a fraction of the normal potential life of the structure, deterioration of the strength characteristics proceeded to the point where disastrous failure of the engineering works occurred. Moreover, analyses of the failure conditions indicate that the shearing stresses mobilized on the failure surface were considerably less than can be predicted by conventional determination of the shearing strength of these soils.

Several other cases of slides in similar types of clay shale have come to the attention of the author in which the same difficulties have arisen in attempts to analyze the conditions. At two of these comprehensive soil investigations were undertaken. These two, known as the Dunvegan and Little Smoky River slides, have been reported in detail elsewhere (1).

The Dunvegan slide involved the failure of a highway embankment built on a slope of 6.5:1 with an over-all height of 250 ft. The embankment was to be built to a height of 100 ft, but when a height of 70 ft had been reached instability developed as the frost was leaving the ground in the spring of 1959. The surface area within the slice was about 50 acres and 4 to 6 million cubic yards of soil moved.

The Little Smoky River slide involved movement of more than 2 ft in a bridge pier foundation founded on timber piles driven to refusal in the clay shale.

In both these cases conventional stability analyses show that they should have had a substantial factor of safety against movement. Total stress analyses are completely unrealistic using shear strengths from tests that appear to be at all reasonable. Effective stress analyses using shear strength parameters determined from consolidated undrained triaxial tests with pore pressure measurements give somewhat more realistic values for factor of safety. However, to secure an effective stress low enough to give a factor of safety of unity, a magnitude of pore pressure must be assumed that greatly exceeds any realistic value indicated by the subsoil investigations.

THEORETICAL CONSIDERATIONS

It is pertinent to consider possible modifications to conventional shearing strength theories that will permit more realistic stability analyses to be made where these types of clay shales are involved. The concepts of Lambe (2) for the shearing strength of clay soils, in which forces of attraction and repulsion between the soil particles are postulated, appear to offer possibilities in providing a modification to the conventional effective stress stability analysis that will more accurately check the observed field performance of these soil types.

Release of stress in recent geological time on these heavily over-consolidated soils results in elastic readjustments within the soil mass which may result in fissuring and fracturing of the brittle soil mass, particularly at comparatively shallow depths and along river valleys. One effect of this is that water can more readily penetrate the soil, which if it has access to free water undergoes a marked reduction in shearing strength. Lambe's theory postulates that this occurs due to changes in the forces of attraction and repulsion between the soil particles.

The modifications of the inter-particle forces appear to be due to several factors, and are influenced by the types of clay minerals making up the soil particles, the types of exchangeable ions in the adsorbed water films, and the salt content of the free water as compared to that of the adsorbed films. A difference in the salt content between the two results in the development of an osmotic pressure within the adsorbed films, which increases the forces of repulsion between the soil particles. The existence of such osmotic pressures in cohesive soils is taken from physico-chemical concepts, applied to fine-grained soils in an aqueous environment, and the application of these principles to soil masses has been discussed in recent years by a number of writers including Bolt (3), Ladd (4), Moum and Rosenquist (5), Seed, Mitchell and Chan (6), Olson (7), and Scott (8).

From the practical point of view there are several important implications of these concepts. First, because the osmotic pressure exists within the adsorbed films and not in the free water, it is not reflected in piezometric pressure measurements. Second, the increase in forces of repulsion manifests itself as a swelling pressure in the soil. Consolidation tests run so that a measure of the swelling pressure developed is determined, therefore give an indication of the extent to which osmotic pressures can develop in the soil, but this appears to be the only routine laboratory test in the field of soil mechanics which is capable of identifying soils susceptible to the development of appreciable osmotic pressures. Third, an increase in repulsive forces between the particles will result in a decrease in effective stress in the soil mass within the meaning of currently accepted concepts of effective stress.

A characteristic of some of the soils at each of the Peace River Bridge, Dunvegan and Little Smoky River sites, as well as at many other sites in the area of the occurence of these soil types, is that they exhibit high swelling pressures as measured in laboratory consolidation tests. The swelling pressure, however, is not a soil constant. It is affected by a number of factors (8). It is greatly affected by the salt content of the immersing water used in the test, and appears to be a maximum for distilled water. It is significant that surface runoff and snow melt are initially distilled water, and their subsequent salt contents must be taken up by contact with soil. Values of swelling pressures measured in consolidation tests do not usually exceed about 4 or 5 tons per square foot for these soil types, but swelling pressures as high as 35 tons per square foot have been measured. A wide range of values is usually obtained in samples from any particular site and many tests will show no swelling pressure. Appreciable swelling pressures do not appear to be necessarily associated only with montmorillonite clay mineral content, or with soils only of high plasticity. A limited number of tests indicate that maximum swelling pressures are developed with sodium-type soils and those with a high content of sodium-exchangeable ions (7) (9).

From the point of view of stability analyses of slopes in these soils, the most important factor emerging from these theoretical concepts is that the osmotic pressures reduce the effective stress within the soil mass in the zones where such pressures develop. It therefore is expedient to take this fact into consideration by a modification of the conventional relationship for the effective shearing strength of a soil:

$$s = c' + p_e \tan \phi' \tag{1}$$

in which

s = effective shearing strength;
c' = effective cohesion;
ϕ' = effective angle of internal friction; and
p_e = effective normal stress = $(p-u)$;
p = total normal stress; and
u = pore pressure measurable by a piezometer.

Assuming that the osmotic pressures act to reduce the effective normal stress, and that their effect can be estimated from swelling pressures determined in consolidation tests,

$$p_e = p-u-p_s \tag{2}$$

in which

p_s is the swelling pressure estimated from consolidation tests.

Eq. 1 can then be written

$$s = c' + (p-u-p_s) \tan \phi' \tag{3}$$

For the Dunvegan and Little Smoky River slides, for which comprehensive soil test data were available, stability analyses show that for a factor of safety of unity in the slide areas, the required swelling pressure is within the range of measured swelling pressures in consolidation tests on samples from the slide areas (1). Sufficient swelling pressure test data were not available from the Peace River Bridge slide area to permit an accurate estimate of swelling pressure to be made.

PRACTICAL CONSIDERATIONS

Further research on the physical-chemical characteristics of these clay shales, the swelling mechanism, and methods for measuring the swelling forces tending to reduce the effective stress is obviously desirable. A wide field for research has been opened up by the concepts. Present knowledge does not indicate a simple solution to the stabilization of slopes in such materials. The concepts, of course, do not condemn the traditional methods for stabilizing slopes, but they do suggest a change in emphasis in some aspects of the standard practice.

Drainage remains of prime importance in improving stability, but the possible effect of surface runoff of low salt content in inducing high osmotic pressures suggests that major attention should be given to the control of surface runoff. Slope flattening by means of toe loadings is still desirable, provided it does not block drainage. However, slope flattening by excavation of the upper portion of the slope may precipitate instability if it results in more ready access of water, particularly surface runoff, to the subsoil. The most objectionable practice with these soil types is the installation of drainage systems to relieve anticipated pore pressures if these are installed so that surface runoff has access to the system, or if it is possible for water to back up into the system from a river at high flow stages.

ACKNOWLEDGMENTS

The information concerning the Peace River Bridge failure and the data from the subsequent investigations were available to the author in his capacity as consultant to Westcoast Transmission Company, Ltd., and Pacific Petroleums, Ltd., the owners of the natural gas facilities at Taylor. Independent investigations were made on behalf of the Department of National Defense of Canada. Technical information was pooled. Data concerning the properties of clay shales were available from the records of R. M. Hardy & Associates, Ltd., consulting engineers. Special stability studies were made by E. W. Brooker (10) under the general direction of the author.

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Slope Failures in Foliated Rocks, Butte County, California

A. L. O'NEILL, Chief, Project Geology Branch, Division of Design and Construction, California State Department of Water Resources

The slope failures discussed occurred during relocation of Highway 40A and the Western Pacific Railroad around the proposed Oroville Dam and Reservoir near Oroville, Butte County, Calif. The relocations are routed along the edge of the Sacramento Valley and through the rugged foothills of the Sierra Nevada, where 8½ miles of the relocations pass through metamorphic rock of the Calaveras group. Most slope failures occurred in a rock type called phyllite, which has been tightly compressed and intensely folded during metamorphism.

Design criteria called for numerous ¾:1 cut slopes, some in excess of 100 ft in height. Benches were used in many of the deeper cuts; however, some large cuts were constructed without benches. Slope stability problems were most serious when cuts were made in directions parallel or near parallel to the direction of foliation of the rock. Very little trouble was encountered when cuts were made at right angles to the foliation. The nature of the failures with respect to geologic structure, the type and consistency of cracking on planes of foliation, and the direction of the displacement along the planes of foliation suggest that stress relief was part of the mechanism contributing to ultimate failure of the slopes.

• PHYSICAL properties of rock and its weathered counterparts can today be determined with a high degree of accuracy in laboratories. Such determinations lead to predictions of the stability of these materials for construction purposes; however, in some cases the rock fails to behave as anticipated. Many failures can be attributed to varying conditions of the rock mass which are not indicated in the sampling and testing. Other than the actual physical properties of the rock, one of the major considerations is the geologic setting. Consideration of the geologic setting should include an analysis of the geologic history of the area and all relationships that the rock and its structure will have with respect to the proposed works, including the state of stress in the rock and the effects that might be anticipated due to rebound or stress relief of the rock mass.

As modern techniques and efficiency for excavating rock and its weathered counterparts increase, it becomes economically feasible to design progressively and construct deeper cut excavations for highways, railroads, and major aqueducts. Obviously, the deeper the excavation, the more critical becomes the cut slope design, because the economy depends on the optimum angle of slope. Mining companies, engaged in large strip mining operations, are keenly aware of this fact because many millions of yards of excavation can be saved by designing the optimum cut slope for their deep pits.

To be able to design deep cut slopes in rock properly, it is becoming increasingly apparent that knowledge of the factors contributing to stress relief must be increased and that these factors must be considered in the investigations and design. The relationship of the geologic setting and the phenomenon of stress relief was well illustrated by slope failures that occurred during construction of some large roadway cuts in foli-

Paper sponsored by Committee on Landslide Investigations.

ated metamorphic rocks during relocation of Highway 40A and the Western Pacific Railroad around proposed Oroville Dam and Reservoir near Oroville, Butte County, Calif. Oroville Dam, now under construction, will be a 735-ft earthfill structure impounding 3½ million acre-feet of water. The dam is a key unit in the California State water facilities which provide for a large aqueduct system to transport surplus Northern California waters to dry portions of Central and Southern California.

The relocations are routed along the edge of the Sacramento Valley and through the rugged foothills of the Sierra Nevada before joining the original routes above the elevation of reservoir tail water. Approximately 8½ miles of the relocations pass through metamorphic rocks of the Calaveras group, which ranges in age from Mississippian to Permian. Rock types encountered included both the metasedimentary and metavolcanic types. Metasediments included slate, phyllite, schist, sandstone, conglomerate, chert, quartzite, and limestone. Phyllite was by far the most predominant metasediment encountered. Metavolcanic rocks included schistose and massive types of low-grade metamorphism. Vulcanism evidently continued intermittently through the period of deposition of the Calaveras group, because in many areas, beds of metavolcanic rock are intercalated with the metasediments.

Most of the slope failures occurred in the phyllites. These rocks, ranging from slate to schist, are normally moderately hard when fresh and will almost always fracture along well-developed planes of foliation. The depth of weathering is variable and appears to depend somewhat on the steepness of the slope, length of exposure to weathering processes, schistosity, fracturing, and localized mineralogy. The planes of foliation provide access for percolating water which contributes to oxidation and deep weathering of the rock.

There is a predominant northwesterly structural trend in the metamorphic rocks. Except for localized deviations, the average strike of foliation is N35°W and the dip ranges from 60°NE to 60°SW. Both the metavolcanics and the metasediments have been tightly compressed and intensely folded during metamorphism. Geologic mapping, done primarily for information along the railroad alignment, indicates that there is much interfingering of the two major rock types. It has been considered that many of the tongue-like projections represent traces of plunging folds. Folding was intense, as illustrated in some localities by repetition of beds within a few feet. It is very difficult to see or appreciate the intense structural deformation in near-surface exposures because of weathering: however, the deformation in railroad tunnels driven for the relocations was observed. Tunnel mapping also helped to amplify the fact that considerable intrusion of the sediments by volcanic rocks took place before, during, and after folding.

Roadways for both highway and railroad have been constructed to modern standards with numerous large cuts and fills, some in excess of 100 ft. Design criteria called for numerous ¾:1 cut slopes. Benches were established at each 40-ft interval of height for railroad cuts. Benches were also used on highway cuts; however, highway criteria did not require benches at any regular interval. Railroad benches in all cases were 14 ft wide and were sloped to drain along the bench to the end of cut.

As might be expected, stability problems with the ¾:1 cut slopes developed only where the rock structure or foliation was oriented at certain angles with the roadway, or direction of cut. The most stable cut in the foliated rocks existed where foliation was at right angles to the direction of cut. One such cut along the highway alignment was constructed 100 ft high, without a bench, at ¾:1. There has been no trouble or signs of impending failure. On the railroad relocation, a large cut with roadbed width for three tracks was excavated at ¾:1 where the foliation was at approximately right angles to the direction of cut. This railroad cut was 120 ft high at its highest point, had benches established at each 40 ft of height, and penetrated mostly phyllite, ranging from the most strongly weathered at the top to fresh rock at the base of the cut. There have been no slope failures in this large cut and even the shallow soil mantle and the zone of strongly weathered rock (which extends about 30 to 40 ft in depth) have remained stable.

Most difficulties were encountered where cuts were made in directions parallel or near parallel to the direction of foliation. Ordinarily, one would expect failure to occur when or where the dip of the foliation was parallel to or out of the surface of the road cut. However, in this case, most of the failures took place on the side where dip of foliation was near vertical or where dip was into the cut slope.

Failures were always preceded by cracking on the cut faces, along the benches, and later, around the top of the cuts. In most cases, cracking took place slowly and developed into offset planes along the foliation. Apparent displacement amounted to as much as 3 to 4 ft before total failure took place. The time interval between the first evidence of cracking and failure ranged from days to several weeks. Displacement along the planes of foliation was somewhat unusual in that the uphill side of the plane was dropped in relation to the downhill side. This phenomenon was quite spectacular in one of the large cuts where the relative displacement was as much as 4 ft. Displacement continued to increase in magnitude until the rocks could no longer stand, at which time the slope failed by tumbling.

Almost all slope failures took place in strongly to moderately weathered rock. In all cases, the structure of the rock was plainly visible. Basically, the cause of some of the cut slope failures must be attributed to the simple fact that cuts were made at a steeper slope than was stable for the degree of weathering. However, similar materials failed in cut slopes made parallel to planes of foliation, whereas they were stable in cuts made at right angles to the geologic structure. The nature of the failures, with respect to the geologic structure, the type and consistency of cracking along planes of foliation and the direction of the displacement all suggest that stress relief, along planes of foliation, was part of the mechanism contributing to ultimate failure. It is believed that residual stresses existed in the weathered, strongly folded metamorphic rocks, and removal of large amounts of materials from the road cuts relieved the rock mass so that strain took place, allowing cracks to develop in the direction normal to the principal stress, or along the planes of foliation. As strain continued along foliation planes, the weight of the rock and soil above caused settlement, resulting in the apparent displacements. Circular cracking at the top of the cut then formed. Weight and pressure of the mass continued to push against the planes of foliation until total failure took place. Where water saturated the rocks, driving pressures from above were greatly increased, resulting in more rapid failure.

Where dip of foliation was in the same direction as the cut slope, much of the driving force due to weight of overlying materials was directed into the rock and along planes of foliation as the strain took place. Therefore, stress was relieved without the striking offsetting along the planes of foliation. Such slopes reached their maximum stability when cut at approximately a dip slope; however, minor slabbing might have been anticipated. Such a dip slope cut was made at one location along the highway relocation and that slope proved to be stable.

Because most cuts had been made to considerable depth before failures started, it was necessary to reslope to $1\frac{1}{2}$:1 in order to provide room for grading equipment to work. The $1\frac{1}{2}$:1 slopes proved stable in all cases except one large cut where hydrostatic pressure aggravated the problem. The upper portions of that cut, which was originally 110 ft high, had to be resloped from $\frac{3}{4}$:1 to $1\frac{1}{2}$:1 and locally to 2:1. To provide relief of the hydrostatic pressures, 8,425 ft of horizontal drain at a cost of \$33,600 was installed. Resloping of that cut involved 575,000 cu yd of additional excavation at a total cost of \$460,000.

Relationship of geologic structure to stability of roadcuts was well illustrated in construction of these relocations. Also, the possibility of residual stresses being present in shallow and weathered foliated rocks has been suggested. The role of stress relief as a mechanism in contributing to conditions causing slope failure is certainly worthy of much study and cannot be ignored in design of deep cut excavations in rock.

Progress in Rock Slope Stability Research

D. O. RAUSCH, A. SODERBERG, and S. J. HUBBARD, respectively,
Mining Engineer, Consulting Mining Engineer, and Associate Mining Engineer, Kennecott Copper Corporation, Western Mining Divisions Engineering Department, Salt Lake City, Utah

The U. S. Bureau of Mines and Kennecott Copper Corporation are engaged jointly in a research program directed toward the development of scientific tools for the design and control of rock slopes. Scientific methods are being developed and evaluated in the program to measure quantitively the effect of rock structure, stress, strength, and ground water on slope stability.

Stresses are being determined from strain or stress-relief measurements made with a borehole deformation gage developed by the U. S. Bureau of Mines or from piezometer measurements. The strength analyses are based on meticulous study of rock structure and strength, and the effect of rock structure and moisture on the strength of a slope. Superficial study of structure is being supplemented by observations inside boreholes with cameras. The slope strength is being investigated by laboratory testing of drill cores from the study area. These tests are to investigate the strength of the rock formations, the frictional resistance along fractures or fractured zones in the rock, and the effect of moisture on strength.

A successful conclusion of the research program should provide the tools to solve two major problems that are encountered with rock slopes: (a) detecting instability before failure develops, and (b) predetermination of precise, safe-maximum slope angles with known safety factors.

•MANY MODERN highways require the excavation of deep cuts in rock. The highway engineer is confronted with the problem of estimating the slope angles for the sides of a cut. The steepness of the slopes governs the quantity of rock to be excavated and it may be the determining economic factor as to whether a tunnel or cut is used in the construction of a new highway.

The mining engineer is confronted with a similar problem in open-pit mines. In these mines the steepness of a pit slope is of major economic importance because the slope predetermines the quantities and costs of waste material removal necessary to recover ore. The ultimate objective is to make the steepest safe slope possible so that the ore can be recovered with the least amount of waste stripping.

The slope stability varies widely for different areas of a pit. Some pit slopes are as flat as 20° from the horizontal, yet they tend to be unstable. Others stand to heights of 600 ft or more at 45° and show no evidence of instability.

Figures 1 and 2 show the importance of determining the various factors that cause slides so that slopes can be designed without risk of failures. Figure 1 shows a slide that occurred in 1930 in the Bingham Canyon pit of Kennecott's Utah Copper Division. About 8,000,000 tons of broken rock were involved in this slide. Figure 2 shows a 1957 slide at the Liberty Pit of Kennecott's Nevada Mines Division at Ruth. About 2,000,000 tons of broken rock moved in this slide and caused a production stoppage for several weeks.

The relationship between steepening slopes and decreasing waste removal requirements ahead of ore mining is apparent. Past and current pit design practices have

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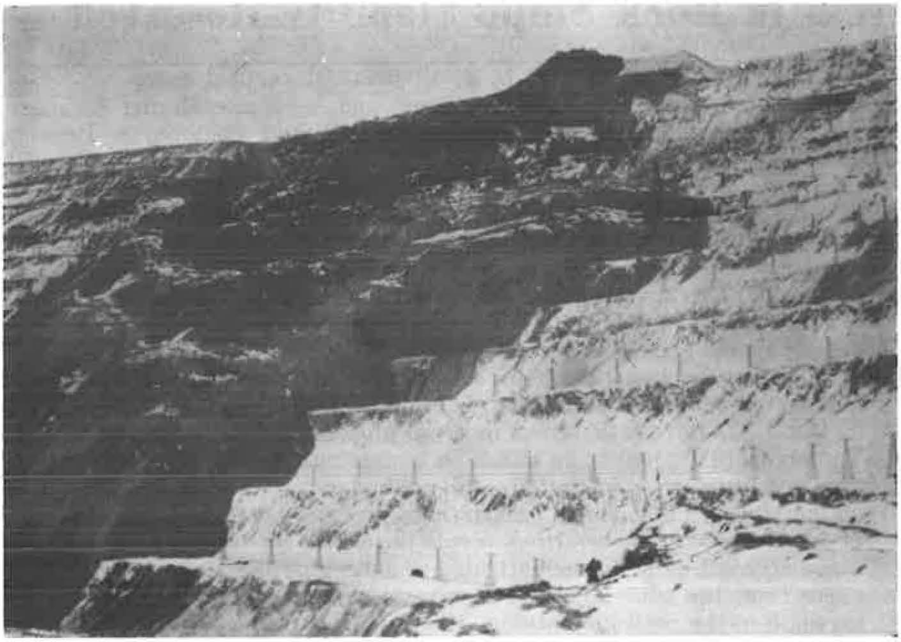

Figure 1. Slide at Bingham Mine.

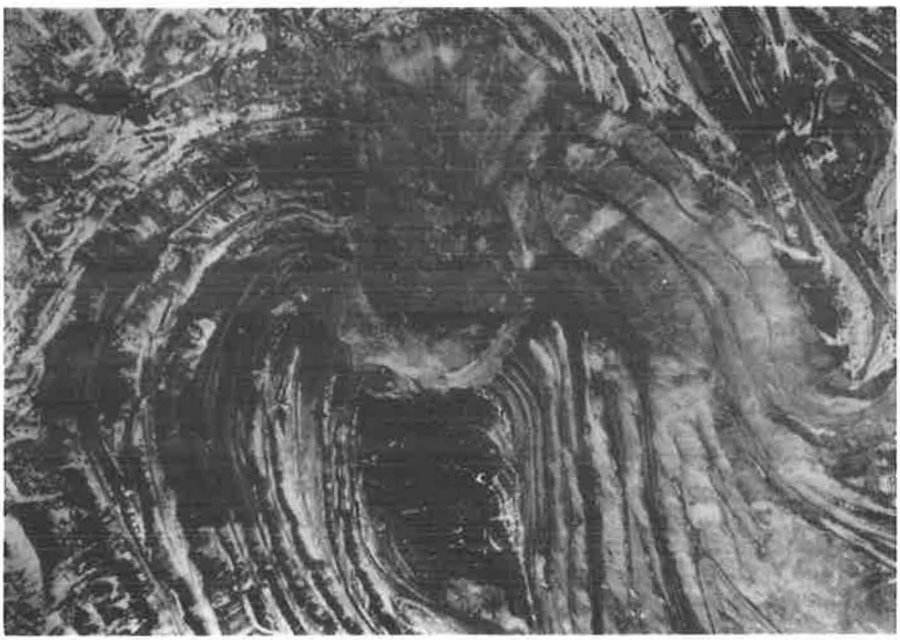

Figure 2. Slide at Liberty Pit.

been and still are based on "cut and try" methods. The two slides shown did result in new empirical design criteria for the pits involved. These experiences, however, are limited largely only to those portions of the pits in which the slides occurred. The remaining sections of the pits are subject to still other stability factors, which are, for practical purposes, unknown or at least unquantified.

Kennecott first sought to employ soil mechanics for determining slope stability in rock on the theory that such methods might apply because the pits are so large that over-all stability might be independent of the size of the largest rock particle, regardless of whether these were several inches or several tens of feet in size. Studies thus far do not support this thinking.

These studies do indicate that the application of soil mechanics is limited in open-pit mines to slopes composed of soil, or to slopes in active slide areas in which the broken, sliding rock mass possesses physical properties similar to soil. Also, as may be expected, the ground water in back of the slope appears to have a major influence on the stability of the slope. But perhaps equally important is the indication that unique combinations of rock structure are the primary cause of rock slope failures, with water seepage through the structures promoting the failures.

RESEARCH PROGRAM

A research program has been undertaken to develop and evaluate scientific methods to measure quantitatively the effect of rock structure and other factors on slope stability. Briefly, it is planned to determine the strength of the west slope of the Kimbley Pit at Ruth, Nev., and to estimate slope angles with safety factors within definite confidence intervals. This is a joint project of the U. S. Bureau of Mines and Kennecott Copper Corporation.

The basic working hypothesis of the program is that the stress in and the strength of a slope govern its stability. Slope failure develops when the slope stress exceeds the slope strength. The slope strength is not merely a function of the strength of the rock constituents, but is limited primarily to the strength of the weakest structural feature and/or lithologic member along which ground movement is unrestrained in the slope.

The west side of the Kimbley Pit has been selected for study in the research program. Mining in this pit was completed several years ago with over-all slopes of 45° and with bench slopes standing at about 54°. The vertical depth of the pit is about 500 ft. The faces of the slopes are generally free of loose rock and the structural features are fairly discernible.

The research program consists of the investigation of (a) regional or field stresses, (b) stresses existing in back of pit slopes, (c) stress changes in slopes caused by mining, (d) tangential slope stresses, (e) slope stresses caused by ground water, (f) rock structure, and (g) slope strength.

Stress Analyses

The stresses in a slope result from the natural forces tending to degrade or flatten it and from the horizontal or lateral forces in the rock in back of the slope. These lateral stresses result from the hydrostatic effect of the rock load and from the regional or residual stresses which may exist in the locality. The lateral stresses produce stresses in the slope which act tangentially to the slope surface. By measuring the absolute residual stresses it is theoretically possible to calculate the magnitude of the important tangential stresses.

Regional Stress Analyses.—Stress-relief measurements have been completed to determine if regional stresses exist in the Ruth district (Fig. 3). Measurement stations were selected because of their elevation and accessibility, and because of the coring characteristics of the rock found in the walls of the openings. The elevations of the stations roughly equal the pit floor elevations in the district.

Evaluation of the data indicates that the stress field in the Ruth district is attributable to the weight of the overburden and is not complicated by tectonic or regional stresses. Calculations were made using a computer in the final evaluation of the data.

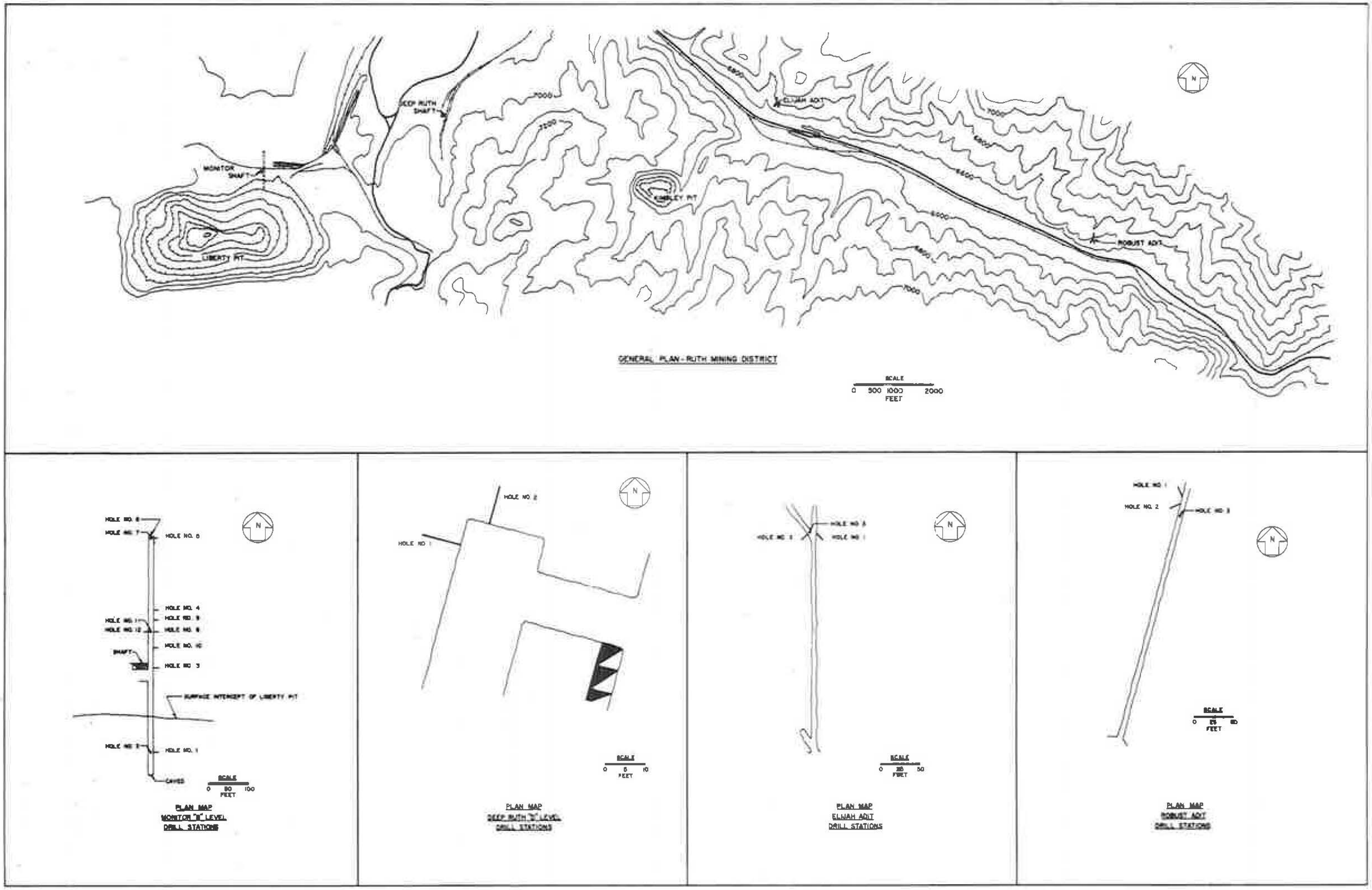

Figure 3. General plan and drill station details, Ruth mining district.

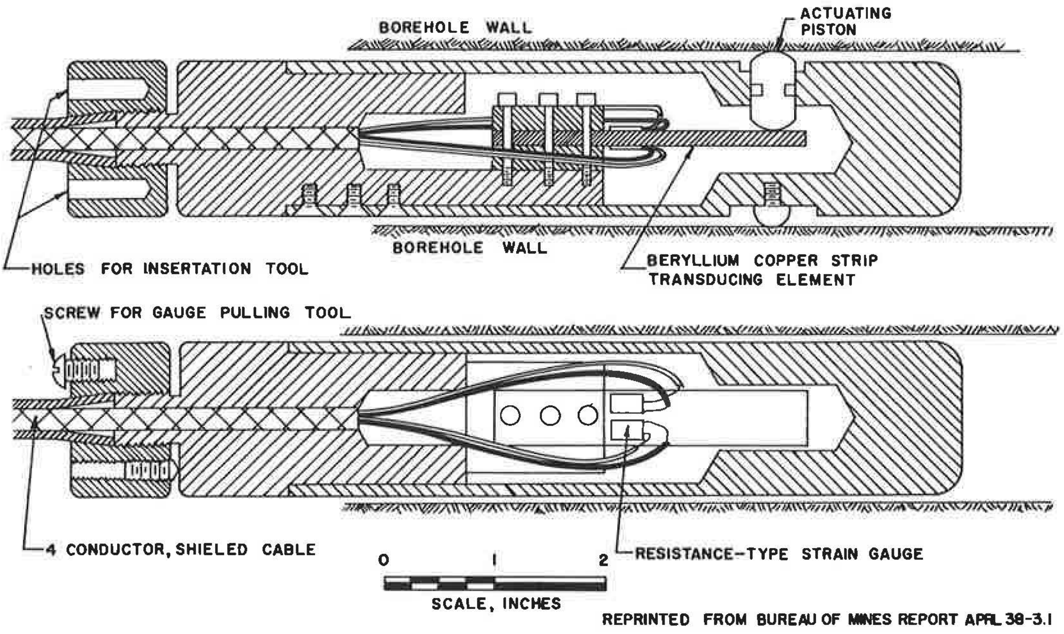

Figure 4. Sections through U. S. Bureau of Mines borehole gage (1).

The information gained will be very useful in the slope angle calculations to be made later in the program.

The strain or stress-relief measurements are made with a borehole deformation gage developed by the U. S. Bureau of Mines (Fig. 4). The gage operates on the principle that the stresses in the rock surrounding a borehole cause it to deform. The deformation of a borehole in a biaxial stress field is shown on an exaggerated scale in Figure 5. The deformation of the borehole is related to the mutually perpendicular applied stresses S and T. If the borehole deformation across three diameters (d_1, d_2, and d_3) is known, the magnitude and direction of the principal stresses, P and Q, and angle θ can be computed (1).

The gage measures the diametral change of the borehole as the surrounding rock is stress-relieved by core drilling concentrically around the borehole (Fig. 6). The increment or decrement of the borehole diameter corresponds to the strain or stress-relief of the rock.

The change in the borehole diameter is measured in three directions as follows: The gage is first inserted in the borehole and oriented at 90°, d_1. The rock surrounding the gage is stress-relieved by core drilling concentrically around the borehole. Coring is stopped

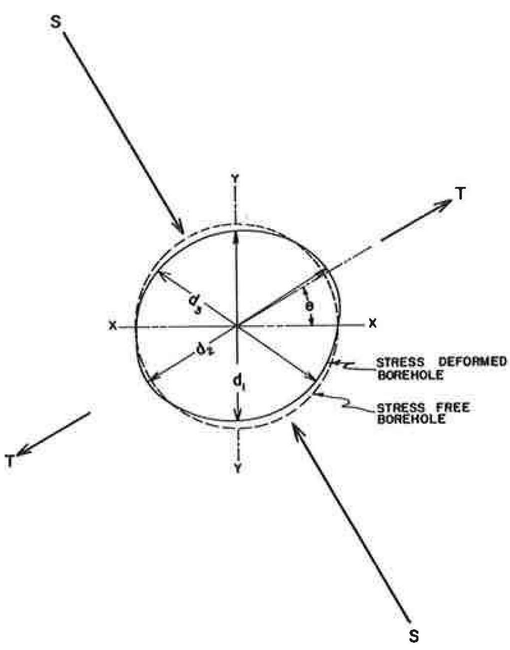

Figure 5. Cross-section of borehole in biaxial stress field.

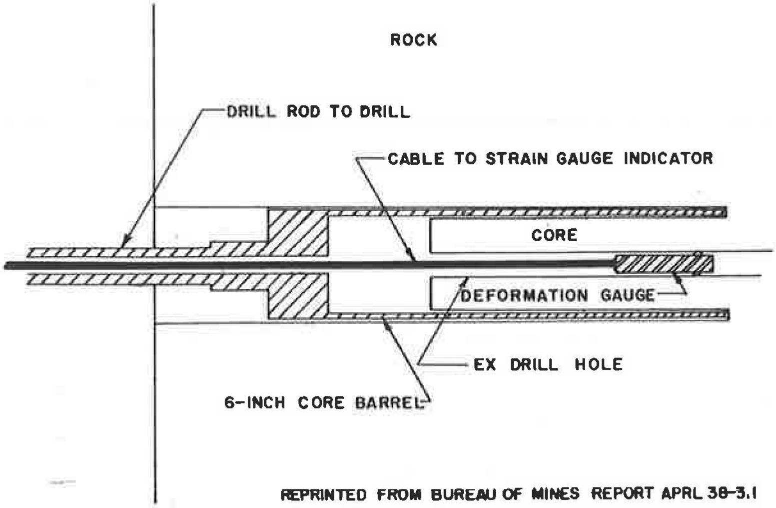

Figure 6. Section of gage and core barrel in borehole (1).

when no further diametral change of the borehole is detected by the gage. Next, the gage is moved past the core depth of the first step, oriented at 30°, d_2, and the stress-relief coring continued. This procedure is repeated for a gage orientation of 150°, d_3. Knowing the change in the borehole diameter in three directions, the magnitude and direction of the principal stresses are computed for the point. Inasmuch as the gage occupies some space, it is impossible to measure the diametral changes in three directions at a point, but if the distances between the three gage positions are known, the stresses may be interpolated for the point.

Deformation measurements are made and the stresses computed for successive points along the borehole until the stresses become constant. The constant stresses are the true residual stresses or field stresses caused by the overburden and are not affected by past mining operations or by the presence of mine openings.

The magnitude and direction of the principal stresses acting on the rock in a plane perpendicular to the axis of the borehole are determined from measurements in the borehole. The principal stresses in three dimensions can also be determined at a point in the rock, but then three mutually perpendicular measurement boreholes are required, which may be accomplished by drilling a hole in the face, floor, and rib at the end of a drift. The regional principal stresses of an area may be interpolated from the residual stresses measured in the three holes.

The transducing element in the Bureau gage is a strip of beryllium copper on which four conventional resistance-type strain gages are bonded to form a four-arm bridge (Fig. 4). The strip is actuated by a piston which senses the diameter of the hole. The strain in the strip changes proportionally to the diametral change, and this strain is measured by a conventional strain gage indicator.

Slope Stress Analyses. —Stress-relief measurements have been made along the entire length of the Monitor "B" level to develop a profile of the stresses existing in the back of the north slope of the Liberty Pit (Fig. 3). Study of the data indicates that the stressed zone, caused by the rock mass of the slope, occurs at an unexpectedly great distance behind the slope face of the pit. These measurements, a first in the field of stress analyses, provide a basis for estimating the area size in back of the slope that must be investigated in the research program.

The distance that the stressed zone occurs in back of the slope suggests that slope failure does not generally develop in this zone in a rock slope. This tends to confirm the hypothesis that slope failure develops in weak structural members closer to the pit face.

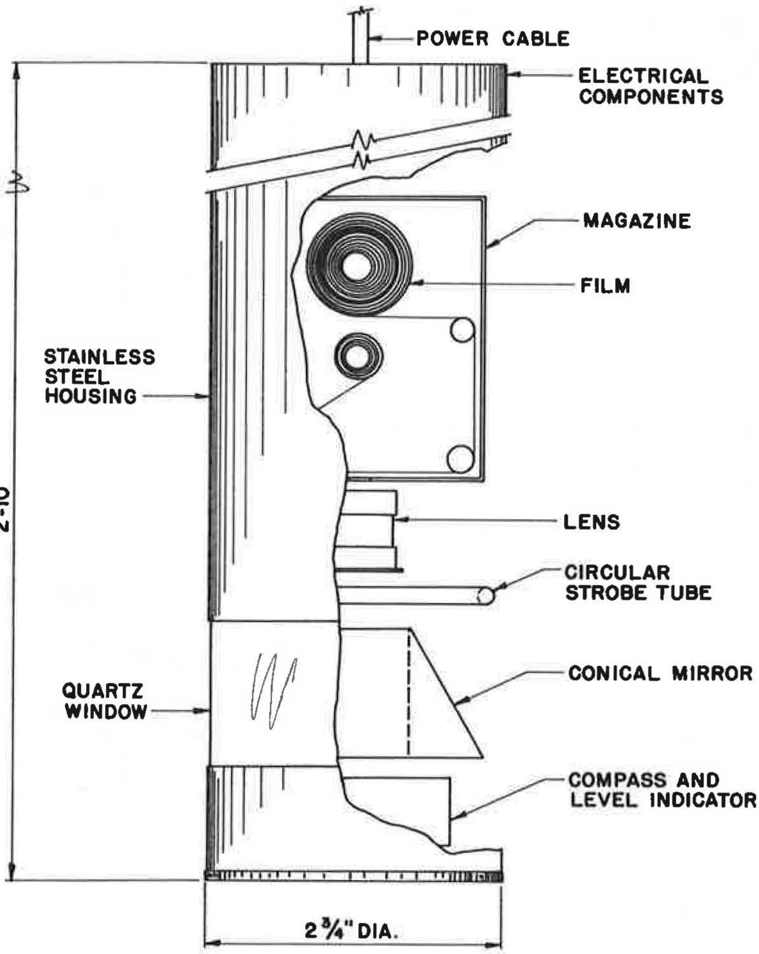

Figure 7. Borehole camera.

Work Stress Analyses.—The magnitudes of the changes in stresses caused by mining, stripping, and changing the degree of slope are unknown. Investigation of these changes could provide information that should be useful in the design of slopes and in the development of instrumentation to warn of impending slope failure.

Measurement of these changes will be made in old underground mine workings oriented roughly perpendicular to slopes being actively mined. This will be done by first determining the absolute stresses present in the slopes from stress-relief measurements made in the walls of the openings. The changes in these stresses caused by mining will be determined by measuring the deformation of the openings.

The stresses in rock surrounding an opening cause it to deform identical to a borehole. The deformation of an opening that occurs under a given stress in rock is proportional to its diameter or size. Therefore, it is much simpler to measure a small stress change in rock from a nominal size opening than from a small borehole.

The deformation changes of the openings in back of pit slopes will be determined by measuring the changes in the distances between pins cemented in the walls of the openings. The sections of the openings at the measurement stations will be made roughly circular, and the pins will be placed in the walls of the circular sections so that d_1, d_2, and d_3 can be measured at each section as shown for the borehole in Figure 5. Only changes in relative stress in competent rock can be determined by this method.

Miscellaneous Stress Analyses. —The existing tangential stresses are being determined from stress-relief measurements in the slopes of the Kimbley Pit. Such stresses develop from removing the lateral confinement of the rock by pit excavation. According to theories of elasticity, these stresses can be determined from the elastic constants of the rock, but it appears advisable that actual determinations from stress-relief measurements are necessary in the initial program. Techniques have been developed to permit stress-relief measurements at depths up to 100 ft.

Piezometers, placed in boreholes at the west end of the Kimbley Pit, will be used to determine the stresses caused by ground water if this appears advisable.

Strength Analyses

The strength analyses are based on meticulous study of rock structure, rock strength, and the effect of rock structure and moisture on the strength of a slope.

Structural Analyses. —The structural analyses are to locate the structural weaknesses in the west end of the Kimbley Pit. Fractures, joints, faults, stratification and foliation planes, chemical planes, orientation of rock constituents, grain boundaries, and other structural features in the rock of the slope are being investigated.

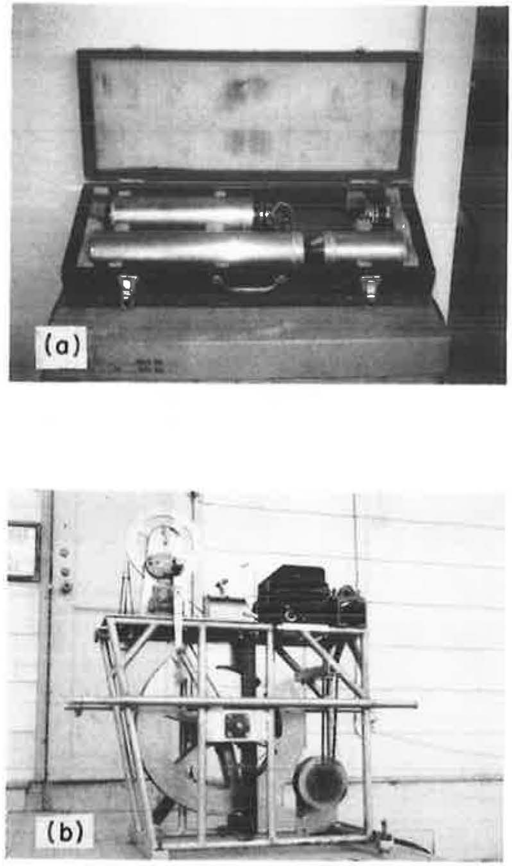

Figure 8. Borehole camera (a) and lowering mechanism (b).

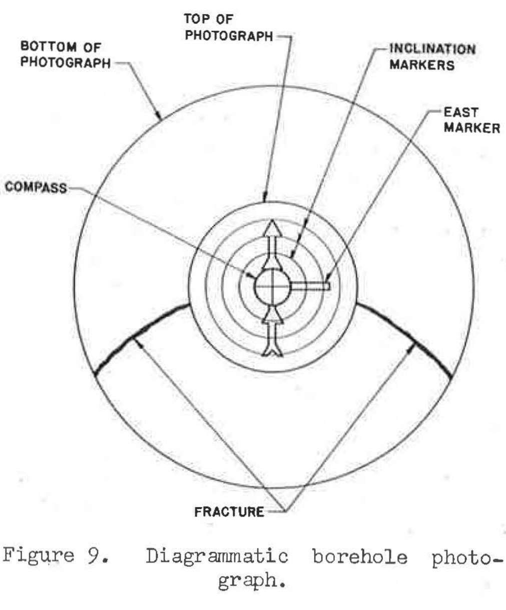

Figure 9. Diagrammatic borehole photograph.

Thousands of structural measurements are being made and interpreted statistically by graphical representations and computers. Superficial study of structure is being supplemented by camera observations inside boreholes. The technique is similar to that described by L. Müller (2) and used by the U. S. Corps of Engineers. W. R. Crane (3) of the U. S. Bureau of Mines first used statistical structural analyses to investigate the stability of the slopes of Kennecott's Bingham Canyon Mine in 1928.

The borehole camera can photograph approximately 75 ft of drill hole without reloading and takes sixteen 360° photographs per foot. Standard 8-mm color movie film is used in the camera. The essential components of the camera are shown in Figure 7. Magnetic north and the camera inclination are recorded on each photograph. Figure 8 shows the camera and lowering mechanism.

A diagrammatic borehole photograph is shown in Figure 9. The radial distance between the top and bottom of the photograph represents approximately $1\frac{1}{4}$ in. of actual hole walls in a $3\frac{5}{8}$-in. diameter drill hole. This distance varies in different diameter drill holes. The east mark is used to eliminate the possibility of viewing the film backwards. The fracture shown in the figure strikes east-west and dips 60° to the south in a vertical hole. The borehole photographs are viewed on a converted microfilm reader at a magnification of slightly less than $1\frac{1}{2}$ times.

Using techniques developed on the job, each fracture photographed is orientated and its true dip and strike are calculated. The data are recorded directly on specially designed computer cards for reduction and evaluation.

Structural data are also collected by visual observations. Large-scale structural features such as faults, major joint systems, and lithologic boundaries are recorded. The data have been plotted on a polar equal-area net and contoured. The results show that the fractures are not randomly scattered but have definite preferred orientations.

Slope Strength Analyses.—The slope strength will be determined by core sampling the study area and by making laboratory tests on these cores. The tests will investigate the strength of the rock formations and the frictional resistance along the fractures or fractured zones in the rock. These tests will investigate also the effect of moisture on strength.

The core sampling will be done by drilling about an 8-in. diameter hole to a prescribed depth by rotary drilling. At the prescribed depth, diamond drilling will be used to recover about a 6-in. diameter core. Several feet of core will be taken at different elevations and spacings in the formations until consistent laboratory data are achieved for specific formations. Special core sampling will be made across areas of particular structural interest. Areas or formations in which core samples cannot be recovered will be classified as areas with minute strength or the formations may be strengthened in the test zone with chemical or cement grout and the actual strength of the formations investigated.

Using the information from the stress analyses, the rock specimens will be loaded to simulate the increased stresses that will develop if the pit walls are steepened. The results of these tests will be of particular importance in determining the strength of the toe of the pit.

OBJECTIVES OF PROGRAM

It is expected that this research program will develop definite conclusions as to the maximum safe slope related to specified safety factors within given confidence intervals. A second program will test these conclusions by redesigning the west end of the Kimbley to the defined ultimate slope and actually mining to it. The accuracy of the safety factor will be determined also by further mining to the slope defined at unity safety factor or to failure.

If successful, the research programs will provide the tools to solve two major problems that are encountered in open-pit mines: (a) detecting instability in a rock slope before any failure develops, and (b) predetermining a precise safe-maximum slope angle with a known safety factor for a rock slope.

A successful conclusion will permit the predetermination and continued maintenance of the optimum safe mining slope. The economies to be realized from mining such a correctly engineered slope will greatly outweigh the cost of testing. These economies will be realized because the maintenance of the optimum pit slope throughout the life of an ore body will constantly permit safe and efficient mining under slopes of known stability, and so eliminate the practice of advanced stripping or excessive stripping to increase the safety factor in areas of uncertain rock strength.

The ultimate goal is to develop methods that can be used in conjunction with the exploration program of a new ore body, such as using the exploratory holes to make the optical soundings. Knowledge of a safe mining slope for a possible open-pit operation would be invaluable in calculating the recoverable ore reserves and in determining the economic potential of the ore body.

It must be emphasized that only the preliminary work has been completed in this research program and that the working hypothesis described in this paper undoubtedly will require changes during the studies. The results gained as the studies are made will dictate the direction and amount of work required in a specific study of the program.

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