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Analysis of Landslides

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Analysis of Landslides

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Determining Corrective Action for Highway Landslide Problems

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THE PROBLEM of landslides has plagued highway departments throughout the country for many years. For some states, and particularly West Virginia, the damage caused by earth movements represents a major expenditure, one that involves hundreds of thousands of dollars annually. Over 80 percent of the area of West Virginia is located in a landslide-susceptible area. The total number of landslides on the state highways has never been established. However, the writer estimates that this total will approach 1,000 on the 31,000 miles of primary and secondary roads in the state.

The complexities of the landslide problem have very few parallels in highway engineering. The literature on the subject carries numerous references to case histories, but none outlines a systematic, complete approach to the solution of a given problem. The recent bibliography published by the Highway Research Board (1) offers a complete summary of the publications relative to mass movements. The work of geologists on landslides has been and is of considerable value. The classification systems suggested by Sharpe (2) and Ladd (3) assist tremendously in understanding the complicated variety of movements that occur. From the viewpoint of corrective actions, the report by Ladd is perhaps the most comprehensive contained in the landslide literature. Numerous engineers (4, 5, 6, 7, 8, 9, 10, 11, 15) have discussed the application of the theories of Soil Mechanics to the analysis of the stability of a landslide, but there are few details concerning the determination of the effect of a corrective action in terms of stability.

The study that led to the following theory was designed to prepare an approach to the analysis and correction of highway problems dealing with landslides in unconsolidated materials. The primary emphasis was to be towards the correction of existing problems. However, it was felt that the principles should be applicable to the problem of design.

The basis for the study was the writer's experiences in West Virginia, combined with general theories from geology, soil mechanics, and highway engineering. The analysis as advanced is for consideration in the study of all landslides in unconsolidated material, with the exception of those of the nature of fluvial transported material, i.e., the water present is far in excess of normal soil moisture, and the debris is a "relatively small proportion of the flowing mass" (2).

Since one of the primary aims of the study was to consider the applicability of the various corrective measures, the investigation could have been accomplished by a study of existing landslides that have been treated. Such an approach was used by Price and Lilly (12) in 1942. However, a direct study was impossible since as a routine department function there were requests to investigate over 100 landslides during the past three years. Due to a personnel shortage, the demand necessitated superficial analyses but it was decided that the program lent itself to the development of a procedure to evaluate the movements. In addition, it became possible to study the applicability and usefulness of various corrective methods. The theories advanced in the following are not complete for three vital factors remain in the evaluation: (1) observation of those landslides that have been corrected by the methods outlined herein; (2) a more comprehensive study of flow movements,

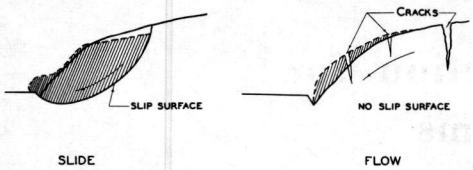

Figure 1. Differentiation between slide and flow (after Sharpe). A slide is a movement of a block of material, whereas flow is entirely internal deformation.

and (3) investigation of less costly methods for correcting landslide problems.

The writer is aware that the analysis is an over-simplification. Extensive study and evaluation is still very necessary, but for the immediate future a working tool is available.

DEFINITIONS

The following definitions will be used throughout. Some of the terms may be argumentative and general, but it is the opinion of the writer that the following are most applicable to the engineering phases of the landslide problem.

Landslides have been defined by Terzaghi (3) as follows: "The term landslides refers to a rapid displacement of a mass of rock, residual soil, or settlement adjoining a slope in which the center of gravity of the moving mass advances in a downward and outward direction." It will be noted that the time element is involved in the definition only by the term "rapid displacement."

The terms slip-plane, slip-surface, and surface of failure will be synonymous and will refer to the surface that separates the mass in motion from the underlying stable material.

Permanent solutions will be defined as corrections with an anticipated life of at least 50 years. An expedient solution will be considered adequate for a period of a few months to 5 to 10 years.

All corrective actions will be classed as one of two types, elimination or control. The actions involving elimination depend generally upon avoiding or removing the landslide. Control methods are defined as corrections which produce a static condition of the landslide for a finite period of time.

While there have been many classification systems proposed, the bases for the classifications have most generally been related to cause and effect of the movement rather than the mechanics. One notable exception is the system proposed by Hennes (7). For a quantitative analysis of a design or correction for a given landslide, the most satisfactory classifi-

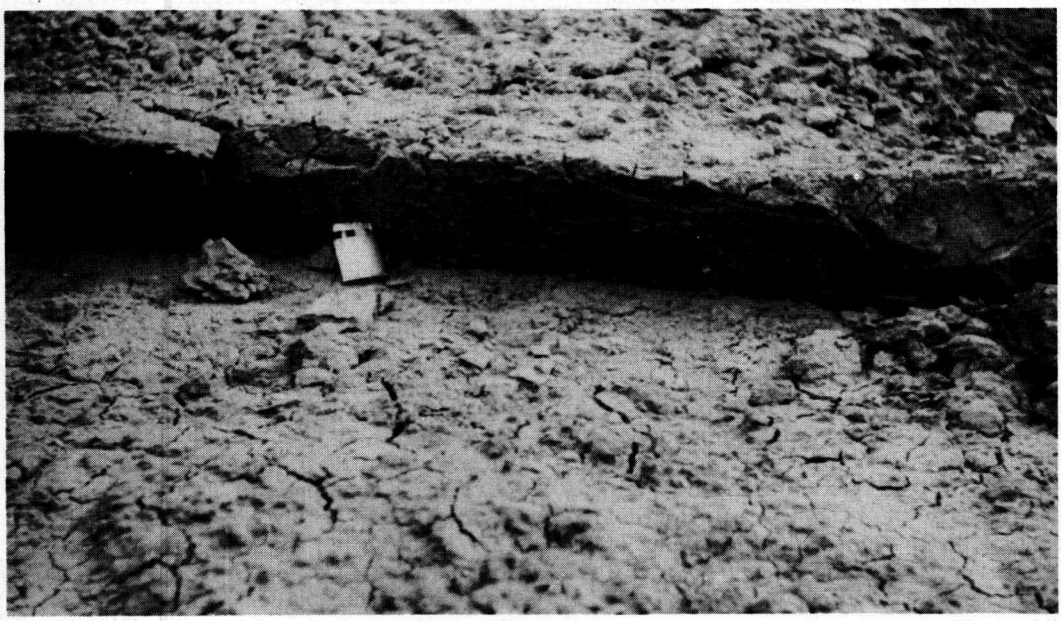

Figure 2. A definite slip-plane, identifying the movement as a slide.

cation is one which differentiates on a basis of the effect of the forces and resistances at work. Thus, the major primary classification would appear to be landslides in consolidated materials, and those in unconsolidated materials. A second primary differentiation would divide the movements into those with a slip-surface and those without a slip-surface. This latter grouping was outlined by Sharpe (2), who termed the former as slides and the latter as flows. The principle is pictured in Figure 1. The movements in flow conditions are the result of internal deformations.

stressed beyond their "fundamental strengths," and as a result, slow but constant internal deformations occur.

BASIC FUNDAMENTALS IN LANDSLIDE ANALYSES

From the observation of landslides in West Virginia, and from a review of the literature on landslides and soil mechanics, the following statements have been outlined by the writer (13) as being fundamental to the analysis of a landslide relative to its correction as a highway problem. It should be emphasized that the statements apply primarily to highway problems and may not be of value from an academic viewpoint or for landslide analyses for other purposes.

Figure 3. Typical flow movement. Note the characteristic roll of the material at the toe. Some movements originate as a flow and develop into a slide.

For the purpose of the following analyses, the term slide (Fig. 2) will be defined as all landslides which involve unconsolidated material in which the movement is along a slip-surface. The terms flow and creep will be defined as those movements which do not have a slip-surface, the movement resulting from internal deformation. A flow (Fig. 3) will be further defined as being caused primarily by excessive water. The term creep will be differentiated in accordance with Terzaghi's concept (3) that failure occurs at a considerable depth due to the load of the overlying material. The layers at the deeper elevations are

1. There are numerous instances where the control of the landslide will not be the best solution. For instances that involve the use of an elimination corrective action that avoids the landslide, halting the movement is not generally a factor in the solution (Fig. 4).

2. Determination of "the" cause of a landslide is not always essential to an accurate solution to a highway landslide problem, and is always secondary in importance to an understanding of the

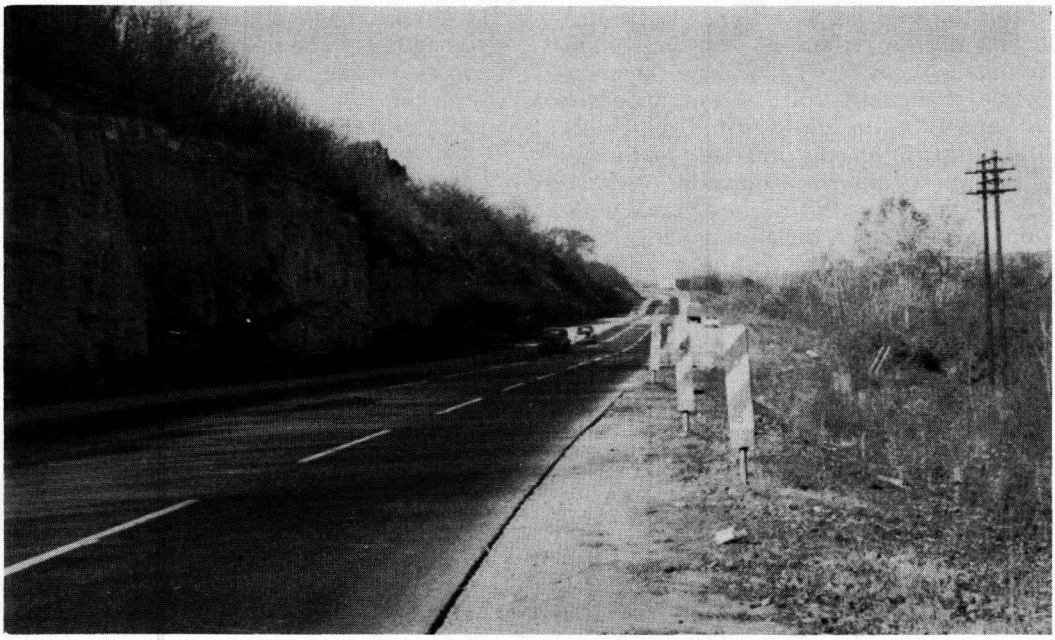

Figure 4. The slide involved in the pictured location is at the right. The problem was solved by shifting the roadway into the stable bedrock at the left of the picture.

mechanics of the movement. The cause of a landslide is often argumentative even after all the available facts have been determined. In many cases, one cause or another may have been the straw that broke the camel's back. Of more importance than the cause, is the realization that increased stability will result by eliminating or minimizing the effect of any contributing factor, particularly that of the effect of the force of gravity.

3. The works of man can measurably accelerate or decelerate the rate of movement of a landslide toward the topographic bottom of the area. Landslides are recognized by geomorphologists as being a major landforming process. The most permanent solutions to control the mass movement will be those of a type that permanently (from a geologic viewpoint) assist nature's resistance.

4. Failure occurs in the soil when the slip-plane is at the contact with the underlying stable bedrock. This observation is valid for all of the instances studied in West Virginia, and was mentioned by Forbes (14) as having been noted in California. Thus, the shear characteristics of the soil at the slip-surface become of primary interest (Fig. 5).

5. For a given landslide problem there is more than one method of correction that can be successfully applied. A common misconception that should be clearly dispelled is that for a given landslide there is one and only one solution. The inference that is undoubtedly intended is that for any given landslide, one method is the most desirable from a consideration of economics, appearance, construction problems, etc.

6. The decision as to the corrective action to be used for a given highway landslide problem is eventually reduced to a problem of economics. This is a statement of an obvious fact, but it is too often subjugated to other considerations. An example that illustrates the point in question would be the case of retaining walls. A wall can be designed sufficiently large to withstand any given landslide. However, a wall design that will be successful may be outside a reasonable range of the economics for a given landslide.

7. Water is a contributing factor in practically all landslides, particularly those involving unconsolidated materials.

Aside from the force of gravity, no factor is more generally present as a contributing factor. The damaging action results from the added weight to the mass, the reduction of shear characteristics of the soil and underlying bedrock (14). Some investigators also state that water produces a lubricating action on the slip-plane. This latter would appear to be a rather unlikely explanation, at least insofar as the mechanics of lubrication are generally accepted.

8. The force of gravity is the sole contributing factor that is common to all landslides. The most obvious basis for a rational analysis is the fact that the force of gravity is the source of all forces tending to cause movement. Until these forces are understood and evaluated, empirical methods are the only available approach.

9. In all mass movements, and just prior to movement, the reactions tending to resist movement are for all practical purposes equal to the forces tending to cause movement. The foregoing statement is an irrefutable fact if the laws of mechanics are valid. Failure to satisfactorily apply a theoretical formula merely means that the method for evaluating the force and the resistance is inadequate. This fact is important since it clearly defines the troublesome features in a rational approach to the mechanics of landslides.

10. The determination of the location of the slip-surface is the most critical factor in the use of a rational or semi-rational approach. Experience has shown that one of the principal limitations on the use of a theoretical approach is the accurate determination of the location of the slip-surface. The problem is involved in both a theoretical office approach and in field examinations. The latter problems are largely due to the lack of a reliable tool that will rapidly, accurately and inexpensively produce the desired subsurface data.

CLASSIFICATION OF CORRECTIVE MEASURES

In order to clarify the analysis, a

Figure 5. The slip-plane developed approximately 1 in. above a layer of stable shale. The scar at the left of the picture developed as the thin layer of clay dried and cracked.

classification was suggested for various corrective measures commonly used in highway landslide problems. It will be recalled that the fundamental difference lies in whether the method involves elimination or control. The following is a detailed classification of the most common corrective measures currently in use. The basis of the classification is the similarity of the analyses within a given group. More details on the methods are given in Appendixes B, C, and D.

I. Elimination methods
   A. Relocation of structure - complete
   B. Removal of the landslide
      1. Entire
      2. Partial at toe
   C. Bridging
   D. Cementation of loose material - entire
II. Control methods
   A. Retaining devices
      1. Buttresses
         a. Rock
         b. Cementation of loose material at toe
         c. Chemical treatment - flocculation - at toe
         d. Excavate, drain and backfill at toe
         e. Relocation - raise grade at toe
         f. Drainage of the toe
      2. Cribbing - concrete, steel or timber
      3. Retaining wall - masonry or concrete
      4. Piling - steel, concrete or timber
         a. Floating
         b. Fixed - no provision for preventing extrusion
         c. Fixed - provision for preventing extrusion
      5. Tie-rodding slopes
   B. Direct rebalance of the ratio between resistance and force
      1. Drainage
         a. Surface
            (1) Reshaping landslide surface
            (2) Slope treatment
         b. Subsurface (French drain type)
         c. Jacked-in-place or drilled -in-place pipe
         d. Tunnelling
         e. Blasting
         f. Sealing joint planes and open fissures
      2. Removal of material - partially at top
      3. Lightweight fill
      4. Relocation - lower grade at top
      5. Excavate, drain, and backfill - entire
      6. Chemical treatment - flocculation - entire

PRELIMINARY ANALYSIS OF A LANDSLIDE

The foregoing is a lengthy list of methods that have been used successfully in controlling or avoiding landslides. Ladd (3) suggested most of those that appear in the classification. The complete list of possibilities should be considered for each landslide at the start of the analysis.

Four factors are required before one can obtain an understanding of the mechanics of the stability of a landslide. These are:

1. The type, character, and topographic description of the underlying, stable bedrock or soil.
2. The location of any seepage strata that are leading into the landslide area.
3. The topography of the ground surface on and adjacent to the landslide. This would include the accurate locationing of the moving area.
4. The types, characteristics, and condition of the soil in and adjacent to the moving area.

Before beginning a detailed field study, a preliminary analysis will be helpful. The principal objectives of these initial field and office studies are to classify the movement, to determine the extent of the movement, to determine the need and scope of additional study, and to determine the probable methods of correction that will be feasible.

Fortunately for the highway engineer, numerous landslides can be handled by elimination methods, i.e., the landslide can be avoided or removed. In such

Figure 6. Drilling will occasionally produce excellent evidence of the location of the slip-plane.

cases, a rapid estimate of the costs involved will show clearly the relative economics and general desirability of an elimination method. For those landslides that cannot be typed as one to be eliminated, an estimate is necessary as to what types of control methods are within reason. With experience, it will become increasingly easier to estimate the corrective methods that will be most economical and otherwise desirable. A study of the appendixes that follow will give some indication of the most desirable set of conditions for the various types of corrective measures. The advantage to this initial estimate lies in the savings that can be realized in future field and office analyses.

FIELD STUDY

Where the situation permits, the field study should extend over several months and, in some cases, years. Unfortunately, many highway problems will require an early decision, and extreme effort will be required to delay action until even a superficial analysis can be made. A study that extends over several months differs primarily from a short study in that continuous observations are made of the direction and the extent of the movement, and of the fluctuation of the ground-water table.

The details to be obtained from the field study will depend upon whether a complete analysis has been deemed necessary. For instance, for certain types of landslides and retaining devices, only the foundation conditions of the retaining device will be needed. As a general rule, however, if a stability analysis is necessary, it will be desirable to obtain the complete information indicated in the preliminary analysis.

In obtaining data concerning the subsurface conditions within the moving mass, various types of drilling as well as geophysical surveys have been used. The most important data to be obtained from this subsurface work are: (1) evidence of the location of the slip-surface (Fig. 6); (2) the condition of the soil as to moisture, density, and structure (for future shear tests); and (3) information that indicates direction and type of movement.

STABILITY ANALYSIS

The following stability analysis is a composite of numerous methods that appear in the literature, and is proposed for use in all landslides involving unconsolidated material. It should be pointed out, however, that applicability of the stability computations to flow and creep movements will require more study, particularly with regard to the location of the potential slip surface. However, by increasing the over-all stability (as indicated by a stability analysis), the actual tendency for flow movement should be lessened.

It is relatively easy to select a corrective measure that will produce a beneficial effect on the landslide area. The purpose of the following analysis is to estimate the degree of stability produced by a given method. In addition, the relative merits and costs of several

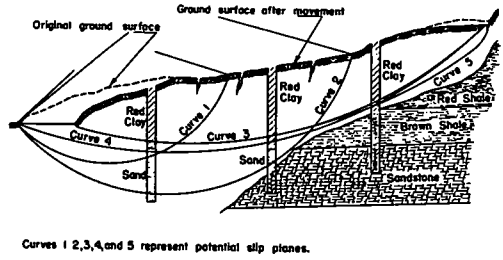

Figure 7. Slide that developed when the toe was cut. Core-drilling located underlying bed-rock. Curves 1 and 2 were established by theoretical formulae. Curves 3, 4, and 5 were adjusted due to layers of underlying stable material.

methods are studied. It is assumed that the resistance to movement equals the force causing movement at the instant of failure. Formulas developed for use in the theoretical soil mechanics are used in the evaluation. Since all of the corrective measures which are considered are analyzed by the same method, the same relative stability should be obtained. The major point of concern is whether the analysis produces an over-design or occasionally an under-design.

Stability analyses of landslides have been applied in two principal ways. If the shear characteristics of the soil are determined, it is possible to estimate the safety factor of the slope. A second procedure is the determination of the average cohesion, or c of the soil at the slip-surface. With the latter method, laboratory tests are not used to determine the shear characteristics of the soil. In either method, it is most desirable to evaluate the landslide under the conditions which existed before the most recent movement. After the determination of the safety factor or the estimation of the shear characteristics of the soil mass, sufficient data are available to estimate the influence of the corrective action.

The method used in West Virginia consists of the procedure involving the estimate of the average c and the following discussion deals primarily with this type analysis. The first step in the stability

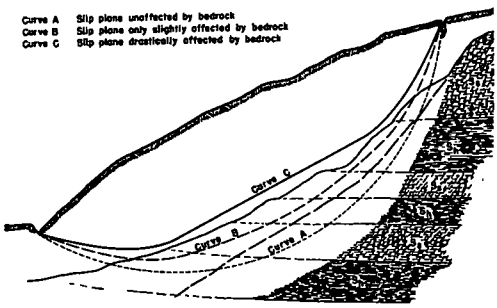

Figure 8. With homogeneous soil, the slip-plane would tend to develop along Curve A. If the area is underlaid by bedrock (as shown in the shaded area) the slip-surface would tend to be as indicated by Curve B. If the bed-rock lies as shown in the solid line, the slip-plane will lie approximately in the position of Curve C.

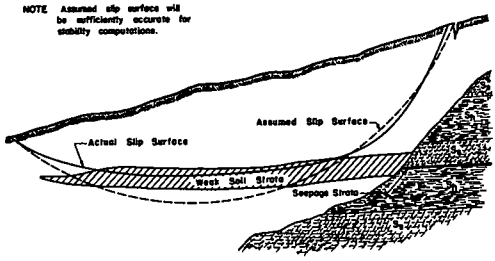

Figure 9. The presence of a weak soil layer will tend to produce a failure within its limits. On occasions, the actual slip-surface will be as close to the theoretical position indicated, and the circle can be used in the computations.

computations is to prepare typical cross-sections parallel to the direction of movement (Fig. 7). The sections should be continued above and below the landslide. On these sections should be plotted all drill information, results of laboratory soil tests, data concerning seepage strata, location of underlying bedrock, surface cracks, structures, and any information considered descriptive of the slide movement. The ground lines both before and after recent movements are very desirable. If the before-movement ground surface is not known, a reasonable estimate will be helpful. The most dangerous sections should then be selected for the initial study. This section will generally be near the middle of the slide, will have the greatest over-all slope (from toe to top), and the greatest mass of unconsolidated material.

The next step is the most troublesome, and perhaps the most vital. The slip-plane must be drawn in its most probable location. The top and bottom of the slide are generally easily identified, but the intermediate portion will call for careful interpretation of the drill data. Observations throughout the past years have led soil engineers to the conclusion that slopes in homogeneous soils fail along surfaces that can be approximated by a circle (in a two-dimensional analysis). Having the top and bottom of the landslide, two points near or on the slip-surface are known. The third, and controlling, point must be estimated. Theoretical formulas (5) suggest a method for the initial approximation. These formulas are for slopes without sur-

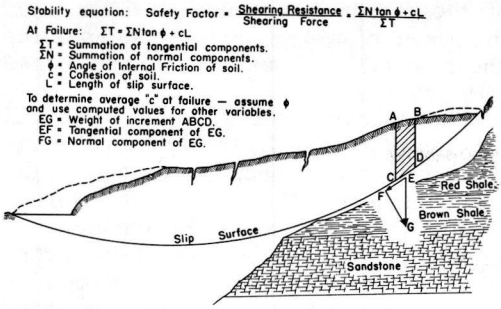

Figure 10. Solution by graphical integration. The total area is divided into increments of the same width as ABCD. Weight of the soil is computed and graphically resolved into its tangential and normal components.

charge and for homogenous materials. However, for the initial approximation, the formulas will be of assistance. The presence of an underlying, firm layer may effect a change in the location of the slip-surface. The change might result in a circle tangent to the layer, two circles connected by a third circle, or two circles connected by a straight line (Fig. 8). The shape of the slip-surface will also be affected by the presence of weak layers (Fig. 9). Taylor (8) has suggested that a circle that approximates a series of curves will be sufficiently accurate.

The drill data will indicate the presence of underlying bedrock or stable soil layers that are in a position to affect the slip-surface. In addition, layers of particularly weak soil can be identified. If there is a question as to the position of the slip-surface, a complete design should be made for each possibility, and the slip-surface that produces the most conservative result should be used.

When the landslide is extensive, slip-planes must be checked for various points up and down the slope (Fig. 7), in addition to the over-all stability. In some cases, several slip-planes will appear reasonable. Each of these should be checked as outlined in the following.

With a reasonable estimate as to the location of the slip-surface, the cross-section of the landslide should be divided into increments, parallel to the direction of movement. Referring to Figure

Figure 11. Photograph of roadway in Kanawha County near Charleston, West Virginia. Note break at right edge of picture.

10, ABCD is a typical increment. The width of the increment is dependent upon the irregularities of the ground surface. Generally, an increment width of 10 to 30 ft. will produce results well within the accuracy of the remainder of the analysis. The weight of the soil in the increment is computed, keeping in mind that the section is assumed to be 1 ft. in width (perpendicular to direction of movement). The weight should be computed for both the original ground surface and the ground surface after movement.

The weight may then be represented by a vector, i. e., a scaled length representing the weight (Line EG). Graphical resolution of this force is accomplished by drawing a line tangent to the centerpoint of the segment of the slip-surface (Line EF). Another line is drawn perpendicular to the tangent at the midpoint of the slip-surface (Line FG).

The intersection of the two lines defines their length. The parallel force is the shear, and the perpendicular force is termed the normal. The resolution of the forces is accomplished for each increment of weight, and the sums of the shear forces (ΣT) and the normal forces (ΣN) are computed.

The forces tending to hold the soil mass in place are (1) the frictional components of the normal forces and (2) the cohesion c of the soil. The forces tending to cause movement are those of shear, seepage, and hydrostatic pressures. There is a diversity of opinion as to the validity of neglecting these latter two forces. Under certain conditions, the hydrostatic forces can be very significant, particularly in cases where cohesionless layers or pockets are present. The effect of the hydrostatic pressure is to reduce the normal forces, and in cohesive soils with a low ϕ value, the change may be insignificant. The seepage forces tend to decrease the normal force as well as to increase the shearing force and the result is significant in the opinion of Taylor (8). In the initial stability analysis, that follows, hydrostatic and seepage forces are neglected, except in their combined effect at

Figure 12. Same slide as that in Figure 11. The toe of the movement is in the middle of the picture.

the time of failure.

A formula that has been proposed for estimating the stability of a slope is the following:

$$\text{Safety factor} = \frac{\Sigma N \tan \phi + cL}{\Sigma T} \qquad (1)$$

where ΣN = the summation of the normal forces in pounds
ΣT = the summation of the shear forces in pounds
ϕ = the angle of internal friction
c = cohesion in pounds per foot
L = length of the slip-surface in feet

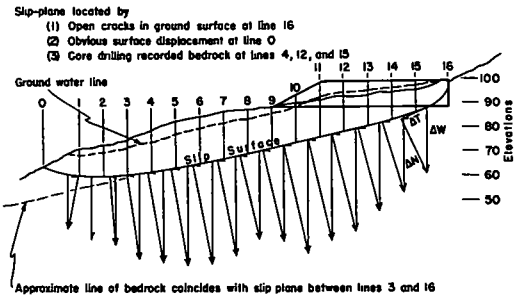

Figure 13. Cross-section of the slide pictured in Figures 11 and 12. The slide has been divided into increments and the computation of ΣT and ΣN is given in Table 1.

Assuming the landslide is at the point of equilibrium between movement and stability (safety factor = 1.0), the following form of the equation is useful:

Shearing force = shearing resistance
or

$$\Sigma T = \Sigma N \tan \phi + cL \qquad (2)$$

It will be noted that the left side of Equation 2 represents the shearing forces causing movement, and the right side is the shearing resistance to movement.

Thus far, the method for obtaining T and ΣN have been indicated. The values of ϕ and c can be determined by shear or unconfined compression tests in the laboratory if desired. Except in rare instances, the laboratory values will not produce a value of 1.0 for the safety factor (Equation 1). This will be true due to irregularities in the soil, to the difficulties in obtaining undisturbed samples, to the problems of laboratory technique,

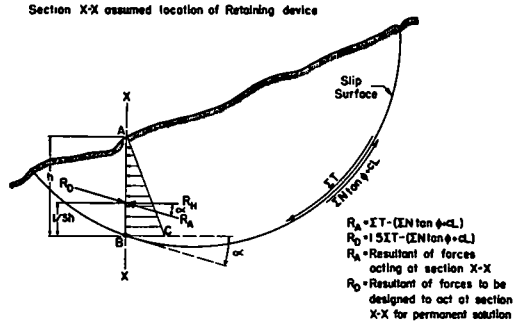

Figure 14. Method for computing the forces acting at a given point in a slide. The value of T and N is determined for the area desired (from X-X to top in this sketch). The difference between the forces causing movement (ΣT) and the resistance to movement ($\Sigma N \tan \phi + cL$) is designated as R_A. The resistance that will produce a safety factor of 1.5 is designated as R_D.

and probably to the effect of hydrostatic and seepage forces. Slopes in nature have been known to be stable even though the safety factor was computed as 0.75. This latter figure would indicate that the shearing resistance was only 75 percent of the shearing force. If the safety factor for a stable slope is less than 1.0, or if greater than 1.0 for an unstable slope, it appears certain that some factor has been disregarded. Numerous examinations have been made of landslide areas, and the computed safety factor was greater than 1.0. Indications were that such computations were based on conditions after the movement. Quite obviously, an area that has moved to a temporarily stable position will show a higher safety factor than 1.0 in its new position. It would appear to be practically impossible to measure the conditions that exist at time of failure. However, if the starting point for the analysis is the ground line prior to movement and Equation 2 is used, the effects of these troublesome variables are accounted for as a part of ϕ or c.

For the following analysis of the stability an estimate is made of the value of ϕ. From Equation 2 it is then possible to compute the average c value needed to obtain an equality between the shearing forces and the shearing resistance. If possible, computations should be carried out for the ground surface con-

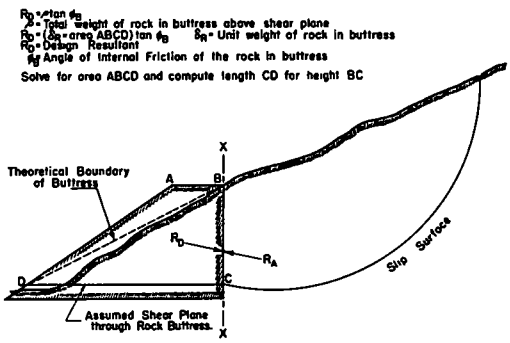

Figure 15. Resistance offered by a rock buttress. The design resultant indicated in Figure 14 must be produced by the shearing resistance of the rock. If the base of the buttress is not bed-rock, a possible shear failure under the buttress must be investigated.

ditions that existed prior to recent movement. From these calculations, the average c value is determined for use in the estimate of the effect of the various corrective measures. The assumption of the ϕ should not lead to serious difficulties. While the c value is very susceptible to varying conditions, the ϕ is relatively constant for a given material. Some investigators have recommended the assumption of $\phi = 0$ for saturated clay soils. This would lead to a more conservative design, since the resistance offered by the normal forces would not be included.

At times it will be necessary to get the range of values for the average c. This will result due to the possibility of various slip-surfaces, and to a range of ϕ values.

APPLICATION OF RESULTS

At the conclusion of the stability analysis the nest step is to estimate the change in the safety factor (or in the ratio represented by Equation 2) that is affected by various corrective measures. The definition of the permanency of a correction was made on the basis of the life of the structure. The importance of a differentiation is in the estimate of the economics involved, i. e., whether or not the structure may have to be replaced. Therefore, to be a permanent correction, the safety factor should be increased by 0.5. This increase can be accomplished by increasing the resistance or decreasing the force. The type of correction governs which of the two (or both) should be changed. Increasing resistance is illustrated in Figure 14 for a retaining device. Unless a significant change can be made in the safety factor, the method is not likely to be helpful on a permanent basis. The use of a corrective action that produces a change of less than 0.5 in the safety factor must be classed as a calculated risk or an expedient. On the basis of Equation 2, a permanent correction should result in the shearing resistance being 1.5 times as great as the shearing force for a permanent correction. If the ratio is less than 1.5, the solution should be considered as an expedient.

The principal difficulty in the follow-up of the stability analysis is the estimate of (1) the additional resistance or (2) the reduction in force that is derived from a specific correction. For elimination methods there are, of course, no problems. Recommended procedures to be used for the control measures are included in the Appendixes C and D. The results thus obtained should not be classed as anything more than an estimate. The degree of accuracy is dependent upon many variables thus far not too well evaluated. In lieu of no other quantitative method, however, the values will be helpful and on the conservative side. In Figure 15, the resistance offered by a rock buttress is illustrated.

In the method involving an estimate of c, it will be interesting to note the relative sizes of the corrections required by applying the upper and lower limits of the range of c values. In many instances, there will be a rather insignificant change in the size of the corrective action needed. For example, a range of 10 deg. in the value of ϕ, made a difference of only 8 percent in the size of a rock buttress. (See Appendix C).

For a given landslide if more than one corrective measure has been indicated as a permanent solution, the final step is an estimate of the costs involved. The decision as to the corrective measure to be employed will be

made on the basis of economy, appearance, effect of the change on driver-safety, or by such other means as established as the policy of the organization concerned.

SUMMARY AND CONCLUSIONS

(1) For highway engineers, the basis for the classification of landslides should be on the mechanics of the movement rather than on cause and effect.

(2) For a given highway-landslide problem there are numerous solutions that can be satisfactorily applied, and the problem can be reduced to a problem in economics.

(3) While the detrimental effect of water has been repeatedly emphasized, the fact that has not been sufficiently emphasized is that the force of gravity is always present as a contributing factor.

(4) By classifying the types of corrective measures in common use, it is possible to clarify the method of analysis of a given landslide.

(5) A preliminary analysis of a landslide should lead to an estimate of the types of corrections to be used. This should reduce the cost of investigating some problems.

(6) The field work should produce all possible data on the location of the slip-surface. The critical factor in the office analysis is the accuracy of the delineation of the slip-surface.

(7) At the moment just before failure the force tending to cause movement is equal to the resistance to movement. The problem is to determine these forces.

(8) The analyses of a landslide should be governed by the basic principle of obtaining a more stable slope than existed prior to failure. At the present time, the best method for estimating quantitatively the relative stability is the formula:

$$\Sigma T = \Sigma N \tan \phi + cL \qquad (2)$$

(9) The forces acting against a retaining device can be estimated as can the resistance offered by the retaining device.

(10) The beneficial effect of any corrective action can be estimated in terms of Equation 2.

(11) The procedure suggested may be an over-simplification in its present form. Observations and evaluation of the corrective measures thus far effected will be necessary.

(12) Considerable research work is necessary to better determine the actual shearing forces and shearing resistances at work in a landslide.

(13) Extensive research is needed to determine an inexpensive method for solving highway landslide problems.

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APPENDIX A

TYPICAL STABILITY ANALYSIS

To present an example of the typical computations in a stability analysis, a landslide in Kanawha County, near Charleston, West Virginia, was selected. Two photographs of the area are included as Figures 11 and 12. The area was core-drilled and cross-sections were taken. The slip-surface was relatively easy to locate. The core shown in Figure 6 was taken from this slide. The underlying bedrock and the obvious extent at the top and at the toe limited the possible position of the slip-plane.

After locating the slip-plane, the area was divided into increments. Referring to Figure 13, the slide area was divided into 16 increments. The width of the increments from lines 1 to 15, inclusive, was ten feet. The two end increments were not an established length. These latter two division lines were set so that the weight of one increment (between lines 1 and 2) would not require a resolution of forces.

The areas of the increments were determined by planimeter. The predetermined unit weight of the soil was multiplied by the area and the total weight of the increment computed. It will be recalled that the cross-section is considered to be 1 ft. in width (perpendicular to the direction of the movement).

The weight of each increment was graphically resolved into a component parallel and another perpendicular to the slip-surface at the midpoint of the width (parallel to the movement) of the increment. Table 1 is a summary of the areas, weights, tangentials, and normals for each of the increments. The ΣT and ΣN for the entire slide area are also shown in Table 1.

The length of the slip-surface was determined to be 180 ft. The range of ϕ values that was considered reasonable was 0 to 10 deg. Referring to Equation 2, all of the variables are now available except the cohesion of the soil. The following summarizes the computations involved in determining c.

$$\Sigma T = \Sigma N \tan \phi + cL, \text{ or} \tag{2}$$

$$c = \frac{\Sigma T - \Sigma N \tan \phi}{L} \tag{3}$$

Assuming $\phi = 0$ deg.

$$c = \frac{53,800 - (285,000 \times 0.0)}{180} = 299 \text{ lb. per ft.}$$

TABLE 1

VALUES OF TANGENTIAL AND NORMAL FORCES IN A TYPICAL STABILITY ANALYSIS

	Increment															
	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9	9-10	10-11	11-12	12-13	13-14	14-15	15-16
Increment Area (Sq. Ft.)	104	120	128	160	176	192	192	188	168	179	219	200	193	168	144	124
Increment Weight (Unit Weight = 110 Lbs. per cu. ft.)	11,440	13,200	14,080	17,600	19,350	21,100	21,100	20,650	18,500	19,700	23,800	22,000	21,200	18,500	15,850	13,650
Tangential Force (Lbs.)	-2,100	0	1,000	2,500	3,200	3,500	4,000	3,800	4,000	4,300	5,000	4,800	5,000	4,500	4,500	3,800
Normal Force (Lbs.)	11,100	13,200	14,100	17,100	19,200	21,000	20,800	20,200	18,100	19,200	23,200	21,800	21,000	18,000	15,300	12,500
Intergranular Force (Lbs.)	5,980	6,110	6,240	7,480	8,160	8,850	8,850	9,230	8,290	8,900	9,600	9,350	9,780	8,730	7,480	7,240

$\Sigma T = 53,800$
$\Sigma N = 285,800$
$\Sigma (N - \mu) = 155,530$

Assuming $\phi = 10$ deg.

$$c = \frac{53,800 - (285,800 \times 0.1763)}{180} = 18 \text{ lb. per ft.}$$

The relatively low c value for this silty-clay soil indicates that ϕ is probably smaller than 10 deg., or that there were strong hydrostatic or seepage forces existing at the time of movement.

For the particular slide in question, assume that the ground water lies as shown in Figure 13. The equation which can be used to account for the hydrostatic pressure is as follows:

$$\Sigma T = \Sigma(N - \mu) \tan \phi + cL, \quad \text{or} \tag{4}$$

$$c = \frac{\Sigma T - \Sigma(N - \mu) \tan \phi}{L} \tag{5}$$

Where $\mu = h \gamma_w l$ = water pressure in lb. at the slip-plane
h = depth in feet from ground water line to slip-plane
γ_w = unit weight of water = 62.4 lb. per cu. ft.
l = length of increment in feet along slip-plane

The values for $(N - \mu)$ are listed in Table 1 as intergranular forces.
For $\phi = 0$ deg., there is no change in c due to hydrostatic pressures since

$$\tan \phi = 0.0$$

For $\phi = 10$ deg., the following is indicated:

$$c = \frac{53.800 - (155.530 \times 0.1763)}{180}$$

$$c = 147 \text{ lb. per ft.}$$

APPENDIX B

ELIMINATION METHODS

The five methods included in this classification are:

1. Relocation of structure - complete
2. Removal of landslide
   a. Entire
   b. Partial at toe
3. Bridging
4. Cementation of loose material - entire

The factor common to these methods is the lack of a requirement for a stability analysis. All of the methods depend upon complete avoidance of the landslide or a complete change of the landslide area. The exception may appear to be the partial removal of the landslide at the toe. Ultimately this will lead to near complete removal. In any event, when carried on as an expedient, no stability analysis is used.

As a very general guide, the following is a list of the elimination methods in order of increasing costs.

1. Removal of landslide - partial at toe
2. Relocation of structure - complete
3. Removal of landslide - entire
4. Cementation of loose material - entire
5. Bridging

I. RELOCATION OF STRUCTURE - COMPLETE

Description - The structure is moved to a location where the foundation is of known stability, either bedrock or stable soil. The grade may or may not be changed, depending upon existing conditions.

Principle Involved - A firm foundation is obtained for the structure.

Best Application - The method is readily applicable to every type of mass-movement. In many cases, the method may prove prohibitive due to excessive cost. The ideal applications are those cases where movements have undermined the structure, and bedrock is located immediately adjacent on the uphill side.

Disadvantages - The cost is usually high if the pavement is of permanent type. Furthermore, the line change may result in poor and unsatisfactory alignment, and finally, the movement is not controlled in event liability is involved.

Method of Analysis - Routine location problem, except particular care should be taken to insure that adequate foundations are available. A complete cost estimate should be made for comparison purposes.

Principle Items in Cost Estimates -

1. Excavation
2. Pavement replacement
3. Right-of-way damages

II. REMOVAL OF THE LANDSLIDE - ENTIRE

Description - All of the slide material is excavated and wasted. This solution applies primarily to movements coming down onto the structure.

Principle Involved - The moving mass that is causing the problem is completely removed.

Best Application - Ideally suited to shallow soil profiles (10 to 20 ft.)and small moving areas (100 to 150 ft. from structure to top of slide). The area above the slide should be stable or worthless, or the question of additional failures should be considered.

Disadvantages - May be too costly for extensive movements. Design care must be taken to insure against undermining the area above, particularly with regard to rockfalls.

Method of Analysis - Normally, the only analyses necessary are for the computations of quantities involved. In cases of questionable stability above the slide area, a stability analysis of the slope above may be required.

Principle Items in Cost Estimate -
1. Excavation
2. Right-of-way damages

III. REMOVAL OF LANDSLIDE - PARTIAL AT TOE

Description - The debris is moved from the area affecting the structure in order to relieve pressure, remove obstacle, etc. Since part of the toe is removed continued movement is inevitable. The method should rarely be used except as an emergency measure. An immediate follow-up with a permanent solution is necessary to prevent future movement.

Principle Involved - The moving mass is excavated so as to permit passage of vehicles, to temporarily relieve pressure against a structure, etc.

Best Application - Very rarely applicable except when movement is down onto structure from above. The method will most often be necessary in instances where the mass has moved against a structure or has blocked a roadway. In instances involving valueless land above, removal of the toe with space provided for future movement may be an economical solution.

Disadvantages - This expedient method does not produce a permanent solution.

Method of Analysis - No analysis is necessary except for quantities involved in a cost estimate. For determining follow-up or permanent correction, a stability analysis of the type required for the permanent solution will be necessary.

Principle Items in Cost Estimate -
1. Excavation
2. Right-of-way damages

IV. CEMENTATION OF LOOSE MATERIAL - ENTIRE

Description - In order to obtain stable material, cement grout is injected into the moving area. This produces a material that has higher shear resistance. In cohesive soils vertical columns are obtained and their effect is that of a system of piling. The same principle is applied when only a portion of the moving mass near the toe is stabilized to produce a buttress.

Principle Involved - The shearing resistance is increased by improving the shear characteristics of the moving mass. In cohesive soils the resisting forces are increased by a transference of load from the moving mass to the underlying stable material.

Best Application - Complete stabilization of the area will not be possible unless the moving mass consists entirely of granular material.

Disadvantages - The principle disadvantage lies in the fact that the method is still experimental and relatively expensive. There is no clear-cut method of estimating the amount of cementing material that will be required. In areas of extensive subsurface seepage, hydrostatic heads may produce flow of entire area unless the pressure is relieved.

V. BRIDGING

Description - The slide area is avoided by a bridge between the two solid extremities of the moving area. Generally, no direct effort is made to control the movement.

Principle Involved - The moving area will not provide a stable foundation for even a part of the roadway or structure. Therefore, firm, unyielding foundations are selected and the area completely bridged.

Best Application - The method is applicable to all types of mass movements. It is particularly suited to steep hillside locations with deep soil profiles, or with bedrock or stable soil at a considerable depth below the desired grade line.

Disadvantages - The main disadvantage is the relatively high cost of the corrective action. In addition, the movement is not controlled in the event liability is involved.

Method of Analysis - Standard bridge design is followed. In most instances, a single span will be desirable due to the lateral thrust that would be applied to a pier constructed within the moving area. Particularly thorough foundation examinations will be necessary to avoid placing the abutments on material that may move in the future.

Principle Items in Cost Estimate -
1. Bridging

Method of Analysis - From a viewpoint of a buttress at the toe, the tone of resistance required can be estimated from a stability analysis. The advantages produced by the cementation will consist of increased shearing characteristics of the soil. The latter values can be estimated by laboratory tests. In instances where hydrostatic heads are involved, the uplift would be a factor in the stability analysis.

The cementation of an entire moving mass is in the category of an elimination method and no analysis is necessary. From a viewpoint of a column action in cohesive soils, the resistance offered by each column can be estimated. Knowing the tons of resistance required for stabilization, it is possible to compute the number of columns required.

Principle Items in Cost Estimate -
1. Equipmental rental
2. Drilling
3. Cement

APPENDIX C

DESCRIPTION OF CONTROL METHODS - RETAINING DEVICES

The corrective measures included in this classification are:

1. Buttresses
   a. Rock
   b. Cementation of loose material at the toe
   c. Chemical treatment - flocculation - at toe
   d. Excavate, drain, and backfill - at toe
   e. Relocation - raise grade at toe
   f. Drainage of the toe
2. Cribbing - concrete, steel, or timber
3. Retaining wall - masonry or concrete
4. Piling - steel, concrete or timber
   a. Floating
   b. Fixed - no provisions for preventing extrusion
   c. Fixed - provision for preventing extrusion
5. Tie-rodding slopes

Further details are available on the following:
Cementation of loose material - Appendix B
Chemical treatment - flocculation - Appendix D
Excavate, drain, and backfill - Appendix D
Relocation - Appendix B
Drainage - Appendix D

Description - A resistance is placed in the path of the moving mass. The resistance is placed somewhere between the structure, or area to be protected, and the toe of the slide.

Principles Involved - Since all retaining devices produce additional resistance to movement, the benefit derived is resisting force that will be added to the shearing resistance (Equation 2).

Advantages - Retaining devices will often permit correction with the least amount of right-of-way damages. In certain cases, only a part of the landslide is brought under control, and a savings is realized over an attempt to control the entire movement. When the area is exposed to stream erosion, the retainer can be designed as a slope protection device.

Disadvantages - Except for floating piles, most retaining devices represent a relatively expensive solution. In addition, except for cribbing, failure of the method will result in a complete loss of the investment involved in the corrective action.

Method of Analysis - Having completed a stability analysis, the point at which the retainer is to be used is selected. The assumed value of ϕ and the average c as computed in the stability analysis are used to obtain the summation of the shearing forces and shearing resistances for the area between the location of the retainer and the top of the slide. This is shown diagrammatically in Figure 14. The summation of shearing forces (ΣT) is multiplied by 1.5. This product will represent the summation of the required shearing resistance for a permanent solution. The actual shearing resistance of the soil is subtracted from the required shearing resistance, and the difference is the force that the retainer must be able to resist without failure.

For all retaining devices, the type and location of stable foundations is a critical factor. In the event bedrock is close, the retainer should be anchored into bedrock. In the event the retainer is placed on soil, the foundation must be below the slip-surface (except for the tie-rodding solutions) and a stability analysis must be made assuming that the slip-surface is diverted to a location below the retainer. In this latter case, the primary benefits of the retainer will result from lengthening the slip-surface, and increased normal forces for $\phi > 0$ deg.

The resistance offered by a retaining device will be the minimum value obtained from the following: (1) friction or shear between the base and the underlying bedrock or soil; (2) increase of normal forces on a slip-surface extending beneath the retaining device (for $\phi > 0$ deg.) and the lengthening of the slip-surface with a corresponding increase in total cohesion resistance; and (3) resistance to shear or to overturning of the retaining device.

The only other factor not considered is the bearing capacity of a soil under a retaining wall. However, there will be few cases where the size or dimensions of the retaining wall will be governed by this factor.

In determining the actual resistance offered by the retaining device, each of the applicable factors mentioned above must be investigated. The original design will be based on the factor that usually controls that particular type of device. The design or location of the correction device must be changed until the minimum resistance offered by one of the three factors is approximately equal to the required shearing resistance.

1. Friction between the base and the underlying bedrock or soil - With the exception of piling, one of the sources of resistance for a retaining device is the friction or shear between its base and the underlying bedrock or soil. The formula for estimating this value is:

$$s = c + p \tan \phi, \quad \text{where} \tag{6}$$

s = shearing resistance of soil in lb.
c = cohesion of the soil at location of slip-plane (lb. per ft.)
p = the weight (direction perpendicular to movement) of the retaining device (lb. per ft.)
ϕ = angle of internal friction between the retaining device and the bedrock (ϕ of the foundation soil).

In cases where the foundation is bedrock, the failure will be at the surface between the bedrock and the base of the retaining device. Thus, for bedrock or granular soil foundations, c = 0 and Equation 3 becomes:

$$s = p \tan \phi \tag{7}$$

For bedrock or granular soil, ϕ will range between 25 and 35 deg. A conservative assumption can be made or laboratory tests can be used to determine the value of ϕ.

For cohesive soils within a buttress or under any retaining device, the cohesion will not be the average c determined for the slide itself. The value for c refers to the material beneath or within the retainer and should be obtained from laboratory tests of undisturbed soil samples.

2. Increase of Normal Forces and of Cohesion Forces on a Slip-Surface Extending Beneath the Retaining Device - This factor will apply only to those retaining devices not founded on bedrock. In addition, for piling the cohesion effects apply but not the increase in normal forces. Another qualification, if the device is placed at a higher elevation than the center of gravity of the slide, the load of the retainer will increase the shearing forces on the over-all slope stability. Thus, full advantages of increasing the normal forces and the total cohesion will rarely be realized unless the retaining device is placed at a lower elevation than the center of gravity of the sliding mass. Finally, an increase of the normal forces will not benefit slides in which $\phi = 0$ deg.

The computation of this factor is accomplished by dividing the cross-section into increments similar to those used in the original stability analysis. For the new slip-surface (recalling that the foundations of the device must be placed below the original slip-surface) the summation of the normal forces will be increased and the length will be greater with a corresponding increase of cohesion resistance.

3. Resistance to shear or to overturning of the retaining device - In order to estimate the resistance to shear or to overturning of the retaining device, it is necessary to know the magnitude, distribution, point and direction of application of the forces acting on the retainer. A suggested method for determining these factors is pictured in Figure 15. From the stability analysis, the required shearing resistance can be determined. The horizontal component of the force can be evaluated by graphical resolution. It is then a reasonable assumption that the force decreases uniformly to a value of zero at the ground surface. There is a vertical component of the required tangential force but the vertical force can be neglected unless the retaining device is placed over a steep portion of the slip-plane. This force does change the direction of the resultant. However, it can be assumed that the resultant acts parallel to the slip-surface. See Figure 15.

DETAILS ON INDIVIDUAL METHODS

1. Buttresses - all types - In each instance, the slip-plane should be assumed to be extending through the buttress. For rock and cementation of loose material at the toe, the slip-surface through the buttress can be assumed to be a straight line extension (Fig. 15). The resistance can be computed from Equation 7. The resistance required at this point is the required tangential force obtained in the stability analysis. Theoretically, a rock buttress should be a triangle that is sufficiently large so as to resist the shear at any point. As a practical consideration, however, the top of the buttress is normally built horizontal for 5 to 10 ft. In Figure 15, a line shows the theoretical limits within which the edge of the buttress should fall. The horizontal widths at various levels are defined by the uniform reduction from the maximum at the slip-plane to zero at the ground surface.

For the buttresses involving soil materials, the resistance of the buttress to shear is computed by Equation 6. The ϕ and c of the material in the buttress should be determined by laboratory tests. For the drainage solution, laboratory permeability tests or field well points should be used to determine the feasibility of drainage. Furthermore, the ϕ and c values should be those obtained from laboratory tests on a sample of the soil under the reduced moisture conditions.

Referring to the example used in Appendix A and to Figures 14 and 15, the following is a typical example of the computations for a rock buttress:

For ϕ = 0 deg., c = 299 lb. per ft., L = 144 ft. for buttress at line 3.

ΣT = 54,900 lb. (from line 3 to 16, inclusive)

ΣN = 247,400 lb. (from line 3 to 16, inclusive)

1.5 x ΣT = 82,350 lb.

cL = 144 x 299 = 43,000 lb.

$\Sigma N \tan \phi + cL$ = 0 + 43,000 = 43,000 lb.

R_D = 82,350 - 43,000 = 39,350 lb.

$R_D = (\gamma_R \times \text{Area ABCD}) \tan \phi_B$

γ_R = 100 lb. per cu. ft.

ϕ_B = 30 deg.

$$\text{Area ABCD} = \frac{39,350}{57.54} = 680 \text{ sq. ft.}$$

For ϕ = 10 deg., c = 18 lb. per ft.

1.5 T = 82,350 lb.

$\Sigma N \tan \phi + cL$ = (247,400 x 0.1763) + (18 x 144) = 46,140 lb.

R_D = 82,350 - 46,140 = 36,210 lb.

$$\text{Area ABCD} = \frac{36,210}{57.75} = 632 \text{ sq. ft.}$$

From the foregoing, the condition of ϕ = 0 deg. gives an 8 percent more conservative figure than that of ϕ - 10 deg., therefore, design the buttress with at least 680 sq. ft. Assuming that the exposed slope of the buttress is on a 1 1/2 : 1 slope (Horizontal : Vertical), and the backslope is vertical:

$$\text{Bases of buttress} = \frac{\text{Area}}{h} \pm \frac{1.5h}{2}$$

If h = 16 ft.

$$\text{Top width} = \frac{680}{16} - 12 = 30.5 \text{ ft.}$$

$$\text{Base width} = \frac{680}{16} + 12 = 54.5$$

The principle items of cost in a buttress are as follows. Not all of the items will be required in every buttress.

(a) Excavation
(b) Backfill (Rock or Soil)
(c) Admixture (Cement or Chemical)
(d) Drainage Pipe
(e) Drilling (for Admixtures)
(f) Equipment Rental (for Admixtures)

2. Cribbing and Retaining Walls - Use is made of standard design methods for the type of wall under consideration. Cribbing should be considered as a gravity-type wall. The magnitude, point of application, and direction of the stresses against the wall will be as indicated in Figure 14.

3. Piling - In order to be fully effective, the piling should extend one-third of its length below the slip-surface. The following is a formula for resistance to shear of the piles (7):

$$s = \frac{A_p \times f_v}{D}, \quad \text{where} \tag{8}$$

s = shearing resistance offered by a pile, in lb. per inch (in a direction perpendicular to the movement)
A_p = cross-section area of the pile in square inches
f_v = allowable stress in shear for the pile, pounds per square inch
D = center-to-center spacing of the piles in inches

A pile should also be checked for the resistance to the soil shearing along each side of the pile. A formula has been suggested by Hennes (7):

$$s = 2\ chd \tag{9}$$

s = shearing resistance per pile in lb.
c = cohesion of the soil, lb. per sq. ft.
h = height from slip-surface to ground surface in feet
d = diameter of the pile in feet

A sufficient number of piles must be available so that the soil shearing resistance or the sum of the shearing resistance of the piles are equal to or greater than the required resultant of the horizontal forces (Fig. 15). The piling will not be subjected to cantilever action until movement has occurred. Due to partial restraint offered by the surrounding material, it should not be necessary to compute the stability of the piles from a cantilever viewpoint unless there is a possibility of movement of the area below the piling.

Referring to the example used in Appendix A, no experienced engineer or geologist would be likely to recommend piling at the location selected for the buttress (Line 3). The computations verify this opinion:

Assuming a 12 in. diameter, timber pile with a cross-section area of 113.1 sq. in., and ϕ = 0 deg., c = 299 lb. per ft.

$$s = \frac{A_p \times f_v}{D} \tag{8}$$

f_v = 100 lb. per sq. in.
$s = \frac{39,350}{12}$ lb. per in.

$$D = \frac{113.1 \times 100 \times 12}{39,500} = 3.45 \text{ in.}$$

$$s = 2\,c\,h\,d \tag{9}$$
$$= 2 \times 299 \times 15 \times 1 = 8970 \text{ lb. per pile}$$

$$\frac{39,350}{8970} = 4.4 \text{ piles per ft.}$$

$$D = \frac{12}{4.4} = 2.7 \text{ in.}$$

For ϕ = 10 deg., c = 18 lb. per sq. ft.

$$s = 2 \times 18 \times 15 \times 1 = 540 \text{ lb. per pile}$$

$$\frac{39,350}{540} = 73 \text{ piles per ft.}$$

$$D = \frac{12}{73} = 0.16 \text{ in.}$$

The obviously low value of 18 pounds per ft. for the cohesion is not a legitimate figure to use unless the material is very fluid. It will be recalled that the average c of 18 lb. per ft. represents the material at the slip-plane. It is not unreasonable to expect a much weaker material at the slip-plane. A more legitimate value for c in Equation 9 is a representative c for the material from the slip-surface to the ground surface at the location of the piles. This could be adequately determined from laboratory tests.

A more reasonable location of piling would be at Line 10 (Fig. 13). The computations follow:

For ϕ = 0 deg., c = 299, 12 in. diameter timber piling

ΣT = 29,600 lb. (lines 10 to 16, inclusive)
L = 66 ft.
1.5 T = 44,400
cL = 19,734
R_D = 24,666
D = 135,800 = 5.5 in.

For average c = 400 lb. per sq. ft. (above slip-plane)

$$s = 2 \times 400 \times 16 \times 1 = 12,800 \text{ lb. per pile}$$

$$\frac{24,600}{12,800} = 1.7 \text{ piles per ft.}$$

$$D = \frac{12}{1.7} = 6.7 \text{ in.}$$

The use of steel or concrete piles would permit wider spacing. The computations would be similar to those for timber piling. However, even a location near the roadway would require very close pile-spacing for a permanent solution.

4. Tie-Rodding Slopes - Resistance will be offered by the piling, cribbing or other retaining device. The remainder of the required resultant must come from the anchorage system. The required resultant (Fig. 15) must be equalled or exceeded by the combined resistance of the retainer and anchorage. The resisting force obtained from the tensile strength of a number of steel bars of a given dimension.

Relative Cost - As a very general guide, the following is a list of the retaining devices in order of increasing costs:

1. Piles - floating
2. Buttress - rock
3. Buttress - excavate, drain and backfill at toe
4. Buttress - relocation - raising grade at toe
5. (a) Buttress - cementation of loose material at toe
   (b) Chemical treatment - flocculation - at toe
6. (a) Cribbing
   (b) Piling - fixed - no provision for preventing extrusion
7. (a) Tie-rodding slopes
   (b) Piling - fixed - provision for preventing extrusion
8. Retaining wall

APPENDIX D

CONTROL METHODS - DIRECT REBALANCE OF RATIO BETWEEN RESISTANCE AND FORCE

The corrective measures included in this classification are:

1. Drainage
   a. Surface
      (1) Reshaping landslide surface
      (2) Slope treatment
   b. Sub-surface (French drain type)
   c. Jacked-in-place or drilled-in-place pipe
   d. Tunnelling
   e. Blasting
   f. Sealing joint planes and open fissures
2. Removal of material - partially at tope
3. Light-weight fill
4. Relocation - lower grade at top
5. Excavate, drain, and backfill - entire
6. Chemical treatment - flocculation - entire

Further details are available on the following:

Relocation - Appendix B
Removal of Material - Appendix C

Chemical Treatment - Flocculation - Appendix C
Excavate, Drain and Backfill - Appendix C
Drainage - Appendix C

Description - The forces that are contributing to the movement are decreased or the natural sources of the resistance to movement are increased. There is no artificial treatment with the exception of chemical treatment.

Principles Involved - The drainage solutions may depend upon the reduction of the shearing forces by the elimination of part of the weight of the moving mass. Drainage may also increase the shearing resistance by increasing c or increasing the intergranular forces (normals) by eliminating hydrostatic pressures. Methods other than drainage either reduce the shearing stresses to a greater extent than the reduction of the normal forces or increase the c value of the soil by increased densities or by treatment of the soil. Chemical treatment may also reduce the water-holding capacity of the soil, which would tend to reduce the shearing forces. Blasting combines the advantages of drainage and the permanent displacement (vertically, upward) of the slip-surface. The slip-plane dispacement by blasting tends to reduce the shearing forces by decreasing the weight of the moving mass, while the beneficial effects of drainage are probably temporary.

Disadvantages - Most of the drainage methods are rather costly, as are excavating, draining and backfilling and chemical treatment. Also, the estimate of the value to be obtained from a drainage solution is extremely difficult. For sealing joint planes, there is a problem of determining whether or not the seepage will develop in another location.

There may be construction problems in installing drainage below the slip-surface in the moving mass. Furthermore, the advantages from drainage of cohesive soil masses may be delayed or may never develop due to low permeability.

Method of Analysis - Having completed the basic stability analysis and having the average c value to be used, the reduction in shearing forces is estimated: for the drainage solution by estimating probable reduction of unit weight of the moving mass, and for removal of material at top, relocation by lowering grade at top, and the light-weight fill. The increase of shearing resistance results from the increase of c value for the following: all drainage solutions (except blasting); excavating, draining, and backfilling; and for chemical treatment. There is an increase of normal force due to eliminating hydrostatic pressures for all drainage solutions, and for excavation, drain and backfill.

The method of analyzing for hydrostatic pressures is a complex field problem of measuring existing ground-water levels (or excess hydrostatic pressures) and estimating probable maximum height. In computations, the effect is shown by Equation 5 or Appendix A. If hydrostatic pressures are to be considered, Equation 5 should be used instead of Equation 1 or 2, in the original stability analysis.

The excess hydrostatic pressures will be particularly troublesome in landslides that contain pockets or layers of free-draining material. It is probable that such pressures are also troublesome in areas where water is relatively free to move down the slip-plane.

Terzaghi (4) points out that in impermeable soils, flash pressures may develop due to heavy rains. Such pressures are relieved before a significant change can be brought about in the water table. He, therefore, recommends a form of piezometric tube to observe these phenomena in the field.

It should be emphasized that the effort to check the effect of hydrostatic pressures is necessary in the procedure outlined herein in order to determine the degree of improvement brought about by drainage solutions. The values obtained by Equations 1 and 2 will be misleading from an academic consideration. However, it is assumed that the most serious condition has been accounted for in the computation of the average c. The

TABLE 2

DETAILS FOR LIGHT-WEIGHT FILL

	Increment						
	9-10	10-11	11-12	12-13	13-14	14-15	15-16
Weight of original soil (lb)	19,700	23,800	22,000	21,200	18,500	15,850	13,650
Increment area (sq ft.)	25	75	105	105	105	105	116
Increment weight (unit weight = 110 lb per cu. ft)	2,750	8,250	11,550	11,550	11,550	11,550	12,700
Weight of soil (lb)	16,950	15,550	10,450	9,650	6,950	4,300	950
Weight of L W fill (unit weight = 40 lb per cu ft)	1,000	3,000	4,200	4,200	4,200	9,200	5,920
Total weight of soil + L.W fill (lb.)	17,950	18,550	14,650	13,850	11,150	8,500	6,870
Normal force (lb)	17,500	18,200	14,500	13,750	10,850	8,200	6,300
Tangential force (lb)	3,920	3,900	3,200	3,260	2,720	2,410	2,920

For area between lines 9 and 16, inclusive, with light-weight fill

$\Sigma N = 89,300$ lb.

$\Sigma T = 22,330$ lb

For entire area with light-weight fill

$\Sigma N = 89,300 + 154,800 = 244,100$ lb

$\Sigma N = 22,330 + 19,900 = 42,230$ lb.

relief of hydrostatic pressure by the installation of a drainage system is not reflected in the stability computations using Equations 1 and 2.

Referring to the example used in Appendix A, for the case of $\varphi = 10$ deg., the average c was computed to be 147 lb. per ft., using Equation 5. If a drain were installed at Line 16, below the slip-plane, and in a position to lower the groundwater table so that it coincided with the position of the slip-plane, the following computations indicate the improvement in stability:

$$S.F. = \frac{\Sigma(N - \mu)\tan\phi + cL}{T} \tag{5}$$

$$= \frac{(285,800 - 0) \times 0.1763 + (180 \times 147)}{53,800}$$

$$= 1.43$$

Thus, the installation of the drain would increase the safety factor by 0.43, which would be sufficient to be termed a permanent solution.

TABLE 3

DETAILS FOR RELACATION - LOWERING ROAD GRADE AT TOP OF SLIDE

	Increment							
	9-10	10-11	11-12	12-13	13-14	14-15	15-16	Σ
Weight of soil (lb)	16,950	15,550	10,450	9,650	6,950	4,300	950	
Tangential force (lb)	3,700	3,270	2,280	2,280	1,690	1,220	405	14,845
Normal force (lb)	16,500	15,100	10,400	9,550	6,750	4,180	870	63,550

For area between lines 9 and 16, inclusive

$\Sigma N = 63,550$ lb

$\Sigma T = 14,845$ lb.

Referring to the same example the value to be obtained from a light-weight fill can be estimated as follows.

Assume that the area between Lines 9 and 16, inclusive, and above the elevation 90.0 is to be removed and replaced with a light-weight material that weighs 40.0 lb. per cu. ft. (unit weight of original soil = 110 lb. per cu. ft.). Table 2 summarizes the change in normal and tangential forces between Lines 9 and 16, inclusive.

Assuming ϕ = 0 deg., c = 299 lb. per ft.

$$\text{S. F.} = \frac{\Sigma N \tan \phi + cL}{\Sigma T} \qquad (1)$$

$$= \frac{(244,100 \times 0) + (299 \times 180)}{42,300}$$

$$= 1.27$$

Assuming ϕ = 10 deg., c = 18 lb. per ft.

$$\text{S. F.} = \frac{(244,100 \times 0.1763) + (18 \times 180)}{42,300}$$

$$= 1.1$$

Thus, the light-weight fill increases the safety factor by 0.1 to 0.27. This would not be sufficient to be considered a permanent correction.

If the grade of the road were lowered to an elevation of 90.0, the following S. F. is obtained (data in Table 3):

Assuming ϕ = 0 deg., c = 299

$$\text{S. F.} = \frac{\Sigma N \tan \phi + cL}{ST} \qquad (1)$$

$$= \frac{0 + (299 \times 180)}{34,745} = 1.55$$

Assuming ϕ = 10 deg., c = 18

$$\text{S. F.} = (218,150 \times 0.1763) + (18 \times 180)$$

$$= 1.20$$

Therefore, lowering the grade would fall slightly short of being a permanent solution. The degree of importance to attach to the ϕ = 10 deg. assumption would be the controlling factor.

Relative Cost - As a very general guide, the following is the list of the methods that modify the shearing resistance or shearing force. This list is in order of increasing cost:

1. Surface drainage - reshaping landslide surface
2. Surface drainage - slope treatment
3. Blasting
4. Light-weight fill
5. Removal of material - partially at top
6. Relocation - lowering grade at top
7. Jacked-in-place or dilled-in-place pipe
8. Subsurface (French drain type)
9. Tunnelling
10. Sealing joint planes or open fissures
11. Excavate - drain - backfill - entire
12. Chemical treatment - flocculation - entire

Use of Field, Laboratory and Theoretical Procedures for Analyzing Landslides

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SYNOPSIS

Mathematical methods for analyzing the stability of many slopes have been available for a number of years. These methods stand ready to be proved, modified, or refuted. There is, therefore, a need for field and laboratory test data taken from actual landslides. Unfortunately, insufficient information of this nature can be found in the literature. There is an abundance of written material concerning landslides available but the bulk of this information is descriptive and is of limited value to the engineer faced with landslide problems.

This paper presents field and laboratory data obtained from three actual landslides. The study was confined to a two-dimensional analysis of a shear-type failure in shallow deposits of unconsolidated materials. The data were used to check the validity of the circular-arc method of slope analysis. The soil strength required for stability, as determined from this method of analysis, was compared with the strength of the soil as measured by laboratory tests. The data are insufficient to indicate definitely the range of applicability of the circular-arc method. However, when combined with similar data from previous studies the results indicate the limited applicability of this approach and point to the area where further study is needed before it can be used to obtain quantitative answers to the problem of prevention and correction of highway landslides.

SINCE THE BEGINNING of time the shape and form of land masses have been undergoing changes. Large portions of the earth have been lifted above their respective surrounding areas while other portions have been depressed. Where these differences in relief occur, even though they may be small, one finds the forces of nature busy at work hewing down or building up these areas to a common level. In parts of the world where conditions are favorable, landslides have been one of the most active forces for changes in appearances of the earth's surface.

Landslides change the surface features of the earth and, in so doing, cause great damage in many areas of the world. It is quite difficult, perhaps impossible, to accurately determine the cost to the public of the slides that occur in a given area in any one year. Even though this problem is not easily resolved, estimates have been made from time to time. About 15 years ago Ladd (1) estimated that the area embraced by western Pennsylvania, southern and eastern Ohio, northern and eastern Kentucky, and western West Virginia suffered an annual damage of about $10,000,000. This estimate is probably based upon the cost of extra construction and maintenance work required by the highways, railroads, and public utilities in this area. This same estimate may, but probably does not, include loss of life, loss of property by individuals, and the wasting of agricultural lands.

Man has always been affected by landslides, but only during the last 100 years has the attention of engineers and geologists been focused on this phenomenon to any great extent. The last 40 years of this period appear to have been the most fruitful from the standpoint of the engineer and geologist. The building of our railroads, the Panama Canal, larger and larger earth dams and our modern highway systems have done much to focus attention upon slides.

Early studies of landslides were concerned primarily with descriptions and methods of classifications. Evidence in

the engineering and geological literature indicates that there has been a recent trend away from this early approach to the landslide problem. The emphasis seems to be shifting away from descriptions and classifications. A greater effort is being made to obtain a better fundamental understanding of the causes and methods of prevention and control of landslides. A better understanding of soil action is being sought. Several mathematical solutions have been proposed and in a few instances attempts have been made to justify one or more of these solutions with field and laboratory test data.

Although this trend appears to be in the proper direction and some progress has been made, much remains to be learned about landslides. Much remains to be learned concerning the shear-strength characteristics of the soils commonly involved in slides. In fact, this phase of the work seems to lag behind the existing mathematical methods of analysis.

PREVIOUS THEORETICAL INVESTIGATIONS

In connection with existing mathematical studies, Carrillo (2) points out that every theoretical investigation in this field assumes that the shearing strength of the soil is governed by Coulomb's empirical equation $s = c + n \tan \phi$ wherein s is the unit shearing strength, c the unit cohesion or unit shearing strength when no confining stress is applied to the soil, n the applied normal stress acting on the surface of failure, and ϕ the effective angle of internal friction. The values of c and ϕ are not constants for any given soil. In the case of clay-like soils they should be regarded as variable coefficients. A detailed discussion of the Coulomb equation and the shearing strength of clay-like soils is beyond the scope of this paper. Nevertheless, the importance of exercising the utmost care in the application of Coulomb's equation cannot be overstressed.

It is also assumed that the soil is homogeneous. Since soils are not perfectly homogeneous no solution can be accepted as entirely reliable.

Carrillo indicates that practically all attempts to apply mathematics to the problem of slope stability can be placed into one of two categories: (1) studies based on the state and distribution of stress in the soil mass at the instant of failure and (2) studies based on the assumption of a potential surface of plastic failure wherein the nature of the stresses in the sliding mass other than along this surface are usually disregarded.

Studies based upon the stress distribution in a slope of perfectly elastic, homogeneous, and isotropic material have been made primarily by French scientists. Resal (3), Frontard (4) and Caquot (5) have made noteworthy attempts to obtain a solution by this method. An exact solution has never been obtained.

Resal based his solution on a generalization of Rankine's state of stress. He worked with a semi-infinite mass of homogeneous material bounded by an inclined plane top surface, subject to its own weight and in equilibrium. The stresses that act on any plane parallel to the top surface are assumed to be vertical and directly proportional to the depth below the top surface. Terzaghi (6) very ably describes this method and offers valuable criticism. Taylor (7) points out that on one important matter the Rankine-Resal method agrees with conditions that are met in the field by indicating that the upper part of the mass is in tension.

Frontard assumed that a Rankine-Resal state is applicable to a slope of finite extent. He conceived a surface of failure for a slope of critical height to be composed of a vertical tension crack, an arc of active failure, and an arc of passive failure. He solved the differential equations for the curved portion of this sliding surface and found them to be deformed hypocycloids. Terzaghi discusses the Frontard solution and points to a number of inconsistencies in this method.

Carillo states that "Caquot abandoned the conception of the infinite slope and assumed, instead, that the stresses at any plane parallel to the slope are uniform and make a constant angle $j < i$ with the normal. This assumption introduces

no improvement (over the Resal-Frontard method), for while the case $j = i$ may be sustained on the assumption of an infinite slope, the case of $j < i$ is an arbitrary conception." The angle i that Carillo refers to here is the angle between the inclined slope and the horizontal.

Others besides Resal, Frontard, and Caquot have attempted to solve this problem from the standpoint of stress distribution in the mass but a method free of contradictions is still unknown.

The second avenue of approach to this problem assumes a potential surface of plastic failure wherein the nature of the stresses in the sliding mass other than along this surface are usually disregarded. This method appears to be more promising at the present time.

Noteworthy contributors to the surface of sliding concept have been Culmann (8), K. E. Peterson of Sweden, Fellenius (9), Krey (10), Glennon Gilboy, Rendulic (11), Taylor (7), Terzaghi (6), and Jåky (12).

Culmann was the first of this group to attempt the analysis of a slope. He assumed that failure would occur along a plane surface through the toe of the slope. However, it is now known that many slope failures occur along a curved surface that sometimes passes below and beyond the toe. The inconsistencies of this method usually lead to results that appear to be unsafe. Today it is of historical interest only.

Sharpe (13) stated that Molitor noted in 1894 that rupture takes place not on a plane but on a curved surface which approaches the form of a hyperbola. Peterson was probably the first to suggest that a circular arc should be used to approximate this curved surface of failure. His assumptions resulted from a study of a quay wall failure in Goeteborg, Sweden in 1916. His observations were supported by the Swedish Geotechnical Commission (14) which subsequently studied a large number of landslides in that country. The circular-arc method of analysis was an outgrowth of these studies. This method has been accepted as satisfactory by many engineers interested in the problem. It requires the use of a number of trial circles to determine the least stable condition. The practical limitations and range of applicability of this method have never been too well defined by experimental data, however.

As a result of the commission's findings, several procedures for analyzing stability based on the circular arc have been advanced. One of these is the method of slices, which was developed by the Geotechnical Commission. A circular arc of failure is assumed and to make the problem statically determinate it is assumed that the forces acting on opposite sides of each vertical slice through the mass are equal, opposite, and collinear.

Another procedure is the ϕ-circle or friction-circle method. This is based upon a method devised by Krey for the analysis of the bearing-capacity problem. It was later applied to the problem of slope stability by Gilboy. It assumes that the failure surface can be represented by a circular arc and that the line of action of the resultant of the friction forces acting on this arc is tangent to a small circle with the same center as the failure circle.

Still another procedure has been proposed by Jåky. He assumed that all points in the sliding mass are on the verge of failure along circular arcs. Carrillo states that "as a consequence of these assumptions, a system of external normal stresses is required at the supposedly free surface."

Rendulic proposed a method in which he assumed that failure occurs along the arc of a logarithmic spiral rather than a circular arc. He assumed that the resultant force acting along this sliding surface is inclined at an angle equal to ϕ from the normal and that it will pass through the origin of the spiral. No additional assumptions were required.

Fellenius seems to have been the first to assume that the angle of internal friction, ϕ, for a sliding mass of saturated clay is zero. This method is becoming widely used, yet the number of cases in which it has been checked against actual failures is still rather small. Skempton (15) discussed this method and cited a number of problems for which it is applicable. He indicated that in cases where the rate of loading of the soil mass is slow enough, ϕ is not

equal to zero and the assumption is no longer valid. He pointed out that clays which are not fully saturated do not have an angle of internal friction equal to zero when tested under conditions of no water-content change. Thus in the case of many man-made embankments, the $\phi = 0$ analysis does not apply. Finally, he indicated that even in the case of fully saturated clays that are tested under conditions of no water-content change, the true angle of internal friction, ϕ, of the clay is not equal to zero. Evidently, there is a difference between the angle of internal resistance, ϕ, and the true angle of internal resistance, ϕ_f, but this difference is not clearly defined by Skempton. In any case he believes that the ϕ equal zero analysis will not, in general, lead to a correct location of the actual failure surface nor will it give a theoretically correct factor or safety if the test values of cohesion are applied to an actual failure surface.

All of these methods are described and discussed in greater detail in present-day textbooks and literature treating the subject of soil mechanics. In addition to these solutions, others have been advanced but in general they are thought to be either too complicated or too specialized to describe herein.

By confining their studies to embankments of homogeneous materials with a constant angle of slope, a level top-surface, neglecting the effects of seepage water, and assuming that the shearing resistance is constant along the failure surface, Taylor and Fellenius have obtained solutions for a large number of cross-sections in terms of the height of slope and physical properties of the soil. Charts are now available in many soil-mechanics textbooks that enable one to analyze quickly slopes of this type. Unfortunately, many of our slides are not of the type assumed and the charts are, therefore, of little value in many instances. Nevertheless, much interesting and valuable information concerning slides was uncovered as a result of the investigation of these two men. By comparing the factors of safety that are obtained when applying the solutions mentioned above (log spiral, ϕ-circle, Culmann plane, etc.) to a particular slope

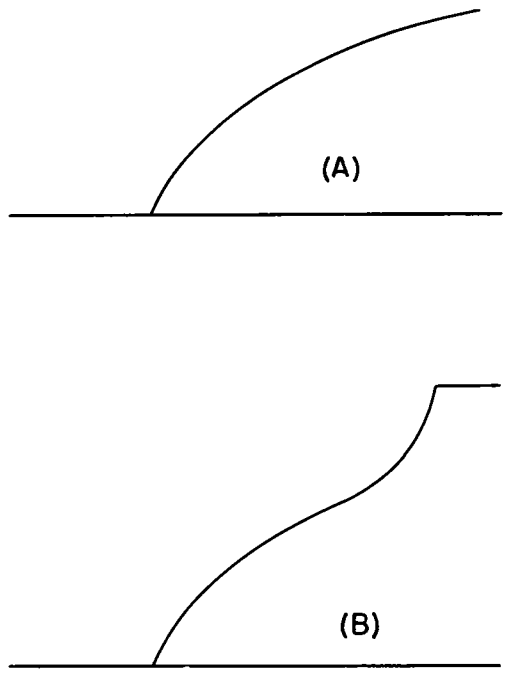

Figure 1. Two profiles of common slope formation of cohesive soils as found in nature.

it became apparent that several of the methods gave approximately the same results. The Resal-Frontard method seemed to produce conservative results while the Culmann plane method gave values that appeared to be unsafe. Even so, the disagreement between the Culmann and the other methods was reduced considerably in the case of steeper slopes. These comparative studies indicated that the ϕ-circle and log spiral methods gave almost identical results. There was even a close agreement between the positions of the two critical failure surfaces as obtained by these two methods.

Taylor also showed that for one slope, at least, the location of the center for the critical circle could be moved about considerably without changing the factor of safety more than 3 or 4 percent. Movement of the center in a direction roughly parallel to the slope was more critical than movement in a direction approximately perpendicular to the slope. These findings are true only for the type of slope studied. However, it appears reasonable to believe that they may also

be true for more complex-shaped sections.

The effects of seeping water in slopes has also been studied by a number of investigators. Several have employed the flow net for this purpose. However, it has been pointed out on a number of occasions that the stress carried by the water at the time of failure is unknown, in spite of what the flow net may indicate.

In conclusion, it might be stated that of the two methods of solution (stress distribution in the soil mass and an assumed surface of sliding) the assumed-surface-of-sliding method currently appears to offer the more promising avenue of approach. Several different solutions employing various sliding surfaces have been developed. For certain types of slides there is evidence that indicates that it matters little whether one chooses the log spiral, φ circle, or slices method to solve a slope stability problem. For practical purposes the final results will all be approximately the same.

Frontard (16), in a slightly different approach to the problem, has suggested that our present method of profiling embankments could be improved. He notes that in every country embankments are constructed by using plane surfaces. At times terraces with plane surfaces are used. He believes that this is proper in case of cohesionless soils. In the case of cohesive soils he believes that it is in direct opposition to everything that can be observed in nature. He thinks that engineers should use more astuteness in determining the shapes of earth slopes. He notes that he has never seen slopes with plane surfaces in hilly areas of clay-like soils. Shown in Figure 1 are two curved profiles that he has encountered in nature. By using embankments with surfaces that have curved profiles rather than plane profiles, he claims that considerably greater heights can be attained without the risk of sliding. He cited examples where profiling methods have been used successfully. In another paper (17) he summarized his mathematical solutions of curvilinear profiles for several slopes. This approach seems to offer possibilities, but further studies are called for.

PREVIOUS FIELD AND LABORATORY STUDIES

Berger (18) recently made a thorough study of this phase of the work. In reporting the results of his work, he noted: "Hundreds of well documented slides are described in the literature. The writer originally expected to find a sufficient number having reliable test data to permit some statistical correlation of the results. However, only 15 slides were actually found having strength data of sufficient reliability to be included in this study. The data for six of these slides were unpublished . . . the slides were all influenced by the presence of a lower critical stratum of soft plastic clay and were generally overlain by stronger clays, or cohesionless material. All slopes were made of nonhomogeneous soils which prevented use of the stability number or the location of the critical center by Taylor's or Fellenius's charts."

As Berger pointed out, nine of these slides had been analyzed and reported in the literature. Nevertheless, he made an independent analysis of each of these slides, and attempted to determine and compare the relationship between the true factor of safety, which he said should be one or less at the time of failure, and the factors of safety that were predicted from the various laboratory test data.

Since there is no standardized laboratory-test procedure available for determining the shearing strength of clay-like soils, Berger encountered many difficulties in his attempts to compare results. In spite of these difficulties Berger did make some comparisons. He found that the factors of safety computed for eight slides for which average unconfined strength data were available ranged from 0.96 to 3.42. Six of these slides showed a very good agreement having factors of safety ranging from 0.96 to 1.25 with an average of 1.14. The other two slides gave values of 1.75 and 3.42. Numerous slickensided surfaces were noted in the soils for the first of these two slides, but no reason could be advanced for the safety factor of 3.42 that was computed for the second slide.

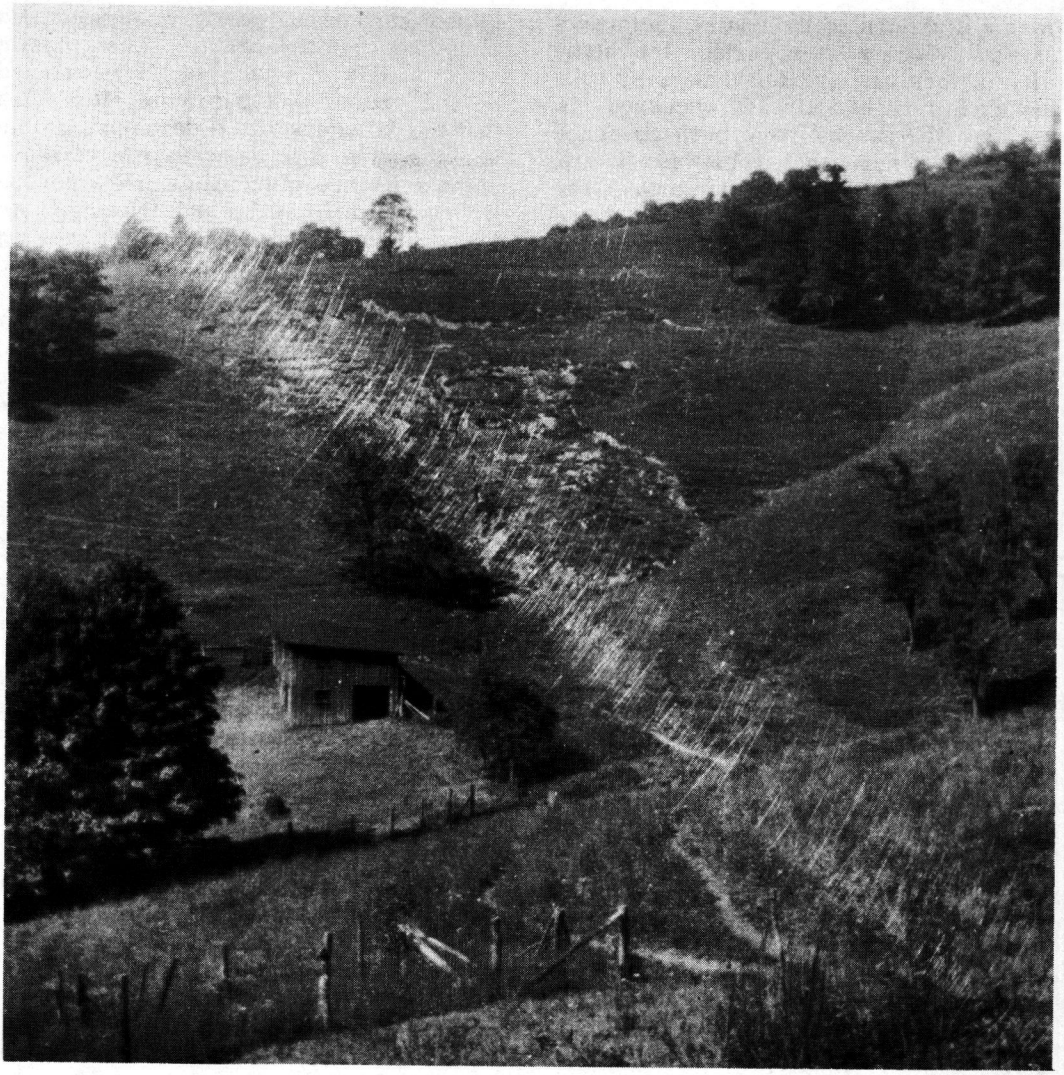

Figure 2. Landslide near Salem, West Virginia.

By using the minimum values of strength as determined by the unconfined compression test, he found that the computed safety factors showed a variation from 0.53 to 1.48 with an average of 0.97.

Factors of safety were computed for four slopes using strength data obtained by the undrained direct-shear test. Three of these resulted in values ranging between 0.96 and 1.11 while the fourth gave a value of 3.81. He said that "no reason can be presented for this extreme value." He indicated that the strength tests were carefully performed in one of the better laboratories in this country.

Triaxial-strength test-data were available for several of these same slides and similar comparisons of stability were made using the data. The results of these comparisons gave safety factors that were, in general, greater than those obtained for the unconfined test.

This one report embodies much of the currently available information concerning this phase of the work. In the opinion of the writer it clearly shows the meagerness of reliable, factual information concerning slope analysis. It indicates that there is considerable scat-

tering of values of calculated factors of safety. Generally speaking, the stability factors are greater than unity even though failure has already occurred. A number of reasons have been advanced for these discrepancies between actual and calculated stability factors. The writer is not prepared to explain these differences. Only after a considerably greater quantity of reliable test data are reported will these questions be resolved.

The type of slide studied by Berger probably differed somewhat from the type analyzed by the writer. The slides that are discussed by the writer occurred in shallow deposits of unconsolidated residual soils. Failures in these soils usually occur along a rather well-defined surface and are often accompanied by a flow. These shallow deposits of soil usually rest above beds of shale, sandstone, and limestone from which they were formed. Quite often these parent beds have been weathered in such a manner that the surface upon which the soils rest is inclined considerably to the horizontal. Water that is carried by the porous beds of sandstone makes its way along this inclined surface of partly weathered and more-impervious shale. The plastic soils in this zone are usually weakened by this water, and it is quite often here that the failure surface develops. In the early spring the upper layers of these same soils are often made more porous by repeated freezing and thawing. An abundant supply of surface water is often available during this same period and there is a tendency for this water to saturate the upper layers of this porous soil. The writer believes that a majority of the slides studied by Berger occurred in rather deep deposits of unconsolidated nonresidual material. Slides in this type of material usually develop along well-defined surfaces and the amount of flow is usually negligible. Spreading sometimes accompanies this type of failure however.

PRESENT STUDIES

Three landslides were studied and analyzed by the writer. One of these slides occurred in a hillside field near Salem, West Virginia. The other two

Figure 3. Landslide on Indiana Route 62. Movement of the embankment undercuts pavement in the right lane.

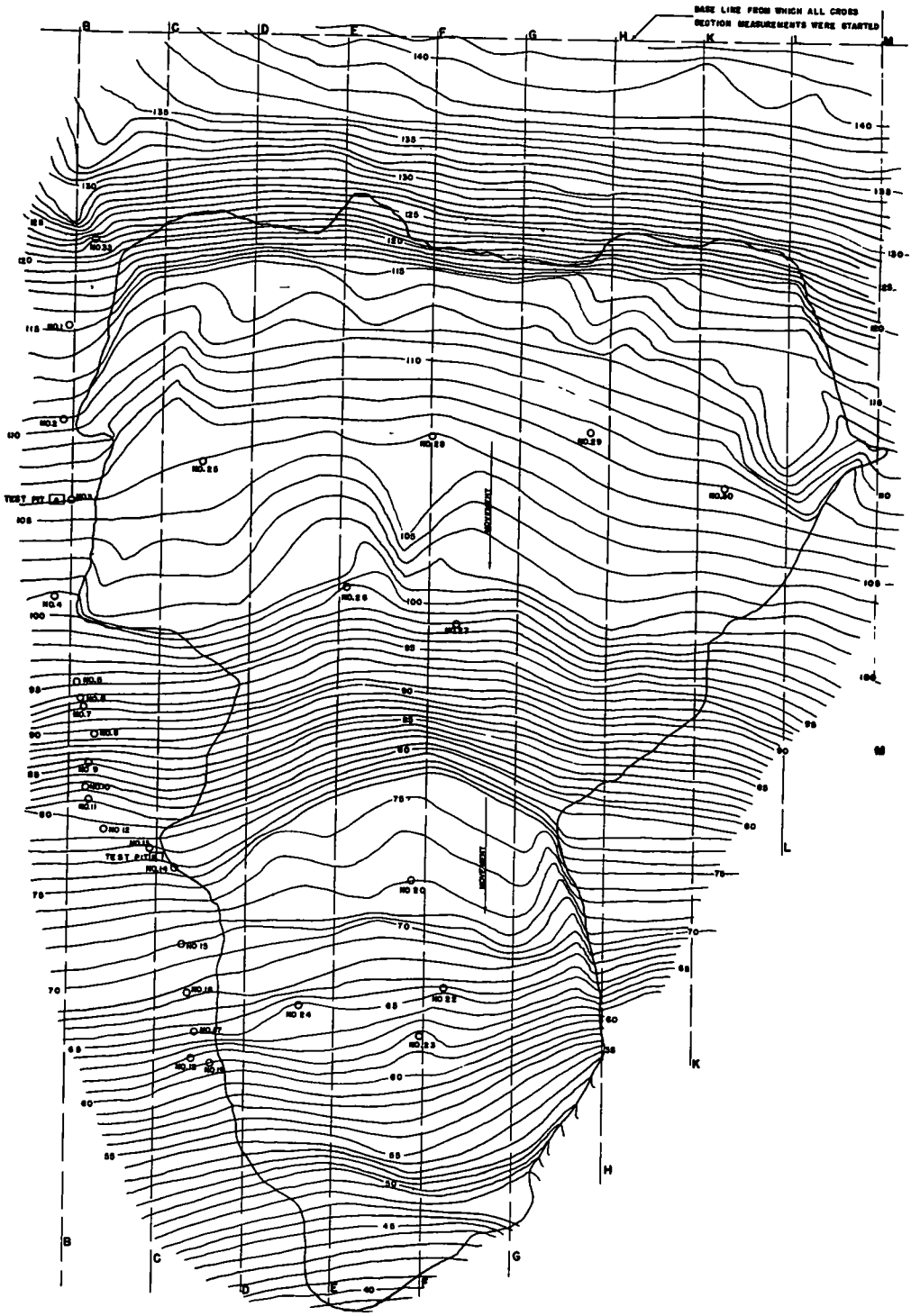

Figure 4. Topography for the Salem slide.

TABLE 1

Slide Location	Liquid Limit %	Plastic Limit %	P I %	Specific Gravity of Solids	Mass Unit Weight lb per cu ft
Salem	57 2	26 9	30 3	2 80	130
English	50 4	20 3	30 1	2 67	126
S H 62	91.0	24 9	66 1	2 60	113

occurred along highways located in Indiana. Photographs of two of these slides and a brief description of the methods that were employed to obtain pertinent data are shown and briefly described below.

Data for the slide near Salem, West Virginia, were obtained by the writer. All subsurface borings were made with a hand auger. Borings into the underlying rock formation were not possible with this equipment. The soil samples obtained from borings were used for preliminary soil identification purposes and for the location of test pits.

Soil samples for testing purposes were taken from test pits along the east edge of the slide. These pits were placed outside of the sliding area. The soil samples from these pits were used for specific gravity, unit weight, Atterberg Limits, and unconfined compressive-strength determinations.

Profiles were plotted and assumed failure surfaces were drawn for several longitudinal sections through the slide. The shearing resistance of the soil was assumed to be a constant and equal to one half of the unconfined compressive strength. The shearing resistance was assumed to be uniformly distributed along the sliding surface. A stability analysis was made for each sliding surface. The strength requirements of the soil for stability were then compared to the available strengths as indicated by the results of the unconfined compression tests.

TABLE 2

Slide	Section	Factor of Safety with Respect to Sliding
Salem	B-B	4 22
	D-D	3 22
	F-F	2 33
	G-G	2 85
	K-K	3 34
English	163+00	4 88
	163+25	5 08
	163+50	5 45
	163+75	7 29
S H 62	1275+00	1 88

Profiles, cross-sections, and boring records for the slide on Indiana State Highway 37 near English, Indiana, and the slide along Indiana State Highway 62 one mile west of Indiana State Highway 145 were furnished by the State Highway Commission of Indiana. Soil samples for testing purposes were taken from test pits adjacent to each of these slides by the writer. The testing procedures and methods of analysis for these two slides were similar to those employed for the Salem slide.

TABLE 3

Slide Location	Factor of Safety with respect to sliding for critical section and critical circle based upon the soil strength as sampled		Factor of Safety with respect to sliding for critical section and critical circle based upon the soil strength of the soaked sample
	Avg Str	Min Str	Avg Strength
Salem	2 33	1 62	1 25
English	4 96	3.85	2 39
S H 62	1 88	1 67	1 08

RESULTS

Shown in Table 1 are certain index properties of the soils for the three slides studied. These are the properties of the soils in the immediate vicinity of the sliding surface. These average index properties show that the soil near the sliding surface for each slide was a highly plastic clay. These clays were only moderately sensitive to remolding.

Shown in Table 2 are the calculated safety factors with respect to sliding required for stability for each of the longitudinal sections. These factors of safety are based upon average values of shearing resistance as determined by laboratory tests. They are minimum factors of safety for that section.

It is interesting to note that for the Salem slide, where data for numerous

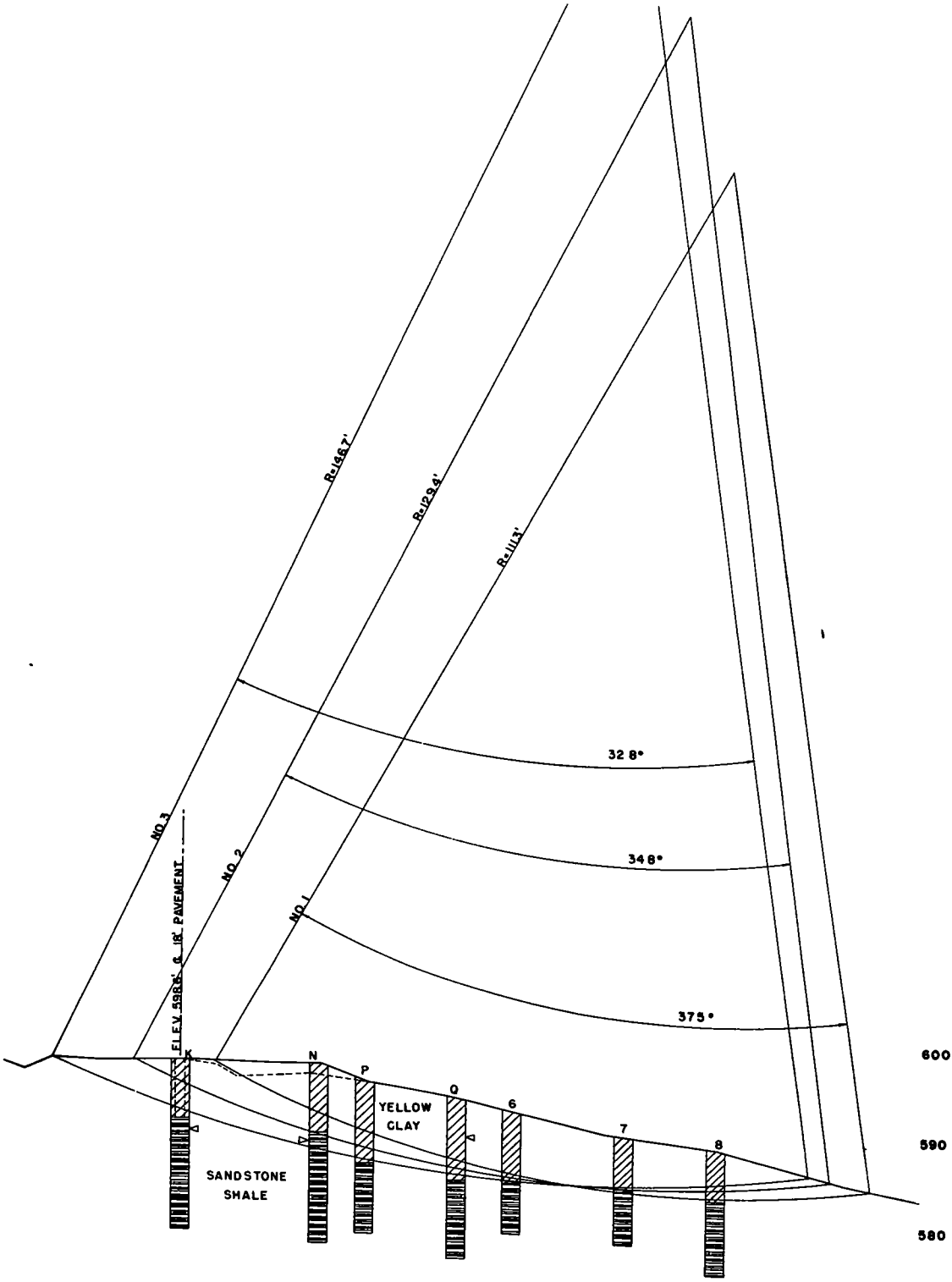

Figure 5. Typical section showing assumed failure surfaces.

sections were available, the factors of safety decrease as we move from the edges of the slide toward the center. These data indicate that the factor of safety varies from section to section. Therefore, it is probable that for this type of slide there is a critical area in which failure commences and from which it then spreads to the adjacent soil mass. Local geological features probably play a large part in the location of these critical areas.

Several circles were analyzed at each station or section. The centers for these circles were often displaced from one another considerably. Nevertheless, in many instances the factor of safety with respect to sliding did not change appreciably for any one set of circles. This is in agreement with Taylor's belief that the exact location of the critical circle is often not exceedingly important.

Shown in Table 3 are factors of safety that were critical for each slide. One column of factors is based upon the average strength of the samples tested. Another column gives factors of safety based upon the minimum shearing resistance of the tested specimens. The last column shows safety factors based upon the shearing strength of a number of samples; these samples were soaked in water since it was apparent that the consistency of the soils as sampled was not the same as one usually encounters for slides of this type at the time of failure.

DISCUSSION OF RESULTS AND CONCLUSIONS

The factors of safety for these three slides based upon the average available shearing strengths are too high. Minimum available strength values give factors of safety that are somewhat lower, but they are still high. This agrees in many respects with the results of work done previously by Berger (18). Soaked samples give strengths that result in factors of safety that are more nearly equal to unity for the Salem and State Highway 62 slides, but are still high in the case of the slide at English. The writer does not advocate the use of soaked samples for strength determinations at this time. Nevertheless, soaking may be justified in cases where the soils involved in a slide change consistency rather quickly and it is evident that the consistency of the sampled soil is not similar to the consistency of the soil in the embankment at the time of failure. The number of slides for which these conditions exist is, of course, limited.

It appears that many slides may always defy mathematical analysis. Nevertheless, the inconsistencies that are present in our current methods of analysis, as evidenced by this work and that of Berger's, may not become apparent until we learn more concerning the shearing strength of clay-like soils, progressive failures, and the effects of seepage forces, tension cracks and impact loads. A better understanding of these factors rather than a new method of analysis holds the key to the embankment problem at the present time.

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