# 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 1s 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 slipsurface. 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. 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 empha-



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

sized 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.

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 slipsurface 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 semirational 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



Curves 1 2,3,4,and 5 represent potential slip planes.

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 same relative stability method, the 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 slipplane would tend to develop along Curve A. If the area is underlind 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 slipsurface will be as close to the theoretical position indicated, and the circle can be used in the computations.

computations is to prepare typical crosssections 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 slipplane 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 slipsurface 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, slipplanes 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 crosssection 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.

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 slipsurface (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 ( $\Sigma$ T) and the normal forces ( $\Sigma$ N) are computed.

The forces tending to hold the soil mass in place are (1) the frictional components of the normal forces and (2) the The forces cohesion c of the soil. 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 The effect of the hydroare present. static pressure is to reduce the normal forces, and in cohesive soils with a low  $\phi$  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:

Safety factor = 
$$\sum \frac{1}{\Sigma T} \frac{1}{T}$$
 (1)

where  $\Sigma N$  = the summation of the normal forces in pounds

 $\Sigma T$  = the summation of the shear forces in pounds

 $\phi$  = 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  $\Sigma T$  and  $\Sigma N$  is given in Table 1.

Assuming the landshide 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  $\Sigma T = \Sigma N \tan \phi + cL$  (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  $\Sigma N$  have been indicated. The values of  $\phi$  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, Section X-X assumed location of Retaining device



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 ( $\Sigma$ T) and the resistance to movement ( $\Sigma$ N tan  $\phi$  + cL) is designated as R<sub>A</sub>. The resistance that will produce a safety

factor of 1.5 is designated as RD.

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  $\phi$  or c.

For the following analysis of the stability an estimate is made of the value of  $\varphi$ . 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  $\phi$  should not lead to serious difficulties. While the c value 1s very susceptible to varying conditions, the  $\phi$ is relatively constant for a given mate-Some investigators have recomrial. mended 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  $\phi$  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 The use of a corrective action basis. 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 correc-If the ratio is less than 1.5, the tion. solution should be considered as an expedient.

The principal difficulty in the followup 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  $\phi$ , 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 driversafety, 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:

 $\Sigma \mathbf{T} = \Sigma \mathbf{N} \tan \phi + \mathbf{c} \mathbf{L}$  (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 cor-

rective 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  $\Sigma T$  and  $\Sigma N$  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  $\phi$  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.

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

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

Assuming  $\phi = 0$  deg.

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

TABLE 1											
VALUES	OF	TANGENTI AL	AND	NORMAL.	FORCES	IN		TYPICAL	STABILI TY	ANALYSIS	

						ncrement										
	0-1	1-2	2.3	9-4	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	513	200	193	168	144	124
Increment Veight																
(Unit Verght = 110 Lbs.	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
por cu. ft.)																
Tangential Force																
(Lbs.)	-2,100	0	1,000	2,500	3,200	3,500	4,000	9,800	4,000	4,900	5,000	4,800	5,000	4,500	4,500	5,800
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	0,850	8,850	9,230	8,290	8,900	9,600	9,350	9,780	8,730	7,480	7,240
	ΣT = 53,	800														
	ΣN = 285	,800														
	Σ(Ν-μ)	= 155, 5	30													

Assuming  $\phi = 10$  deg.

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

The relatively low c value for this silty-clay soil indicates that  $\phi$  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:

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

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

Where  $\mu = h \gamma_w 1$  = water pressure in lb. at the slip-plane

h = depth in feet from ground water line to slip-plane

 $\gamma_{W}$  = unit weight of water = 62.4 lb. per cu. ft. 1 = 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

$$\tan \varphi = 0.0$$

For  $\phi = 10$  deg., the following 1s indicated:

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

$$c = 147$$
 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 eco-nomical 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 1s injected into the moving area. This produces a material that has higher shear resistance. In cohesive soils vertical columns are obtained and their effect 1s 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

## 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  $\phi$  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 ( $\Sigma$ T) 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  $\varphi > 0$  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:

$$s = c + p \tan \phi$$
, where (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. perft.)
- $\phi$  = angle of internal friction between the retaining device and the bedrock ( $\phi$  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:

$$s = p \tan \phi$$
 (7)

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

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 slipsurface (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  $\phi$  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  $\phi$  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 \phi = 0 deg., c = 299 lb. per ft., L = 144 ft. for buttress at line 3.

\Sigma T = 54,900 lb. (from line 3 to 16, inclusive)

\Sigma N = 247,400 lb. (from line 3 to 16, inclusive)

1.5 x \Sigma T = 82,350 lb.

cL = 144 \times 299 = 43,000 lb.

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

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

RD = (\gamma_R \times Area \ ABCD) tan \phi_B

\gamma_R = 100 lb. per cu. ft.

\phi_B = 30 deg.

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

For \phi = 10 deg., c = 18 lb. per ft.

1.5 T = 82,350 lb.

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

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

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

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

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

If 
$$h = 16$$
 ft.

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

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 15 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):

$$s = \frac{A_p \times f_v}{D}, \quad \text{where}$$
 (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
- $f_y^{P}$  = allowable stress in shear for the pile, pounds per square inch

 $\mathbf{D}$  = 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):

$$s = 2 chd (9)$$

....

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  $\phi = 0$  deg., c = 299 lb. per ft.

$$\mathbf{s} = \frac{\mathbf{A}_{\mathbf{p}} \times \mathbf{f}_{\mathbf{v}}}{\mathbf{D}}$$
(8)

 $f_{v} = 100 \text{ lb. per sq. in.} \\ s = \frac{39,350}{12} \text{ lb. per in.} \\ D = \frac{113.1 \times 100 \times 12}{39,500} = 3.45 \text{ in.} \\ s = 2 \text{ chd} \\ = 2 \times 299 \times 15 \times 1 = 8970 \text{ lb. per pile} \\ \frac{39,350}{8970} = 4.4 \text{ piles per ft.} \\ D = \frac{12}{4.4} = 2.7 \text{ in.} \\ For \phi = 10 \text{ deg.}, c = 18 \text{ lb. per sq. ft.} \\ s = 2 \times 18 \times 15 \times 1 = 540 \text{ lb. per pile} \end{cases}$ (9)

 $\frac{39,350}{540}$  = 73 piles per ft. D =  $\frac{12}{73}$  = 0.16 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  $\phi = 0$  deg., c = 299, 12 in. diameter timber piling

 $\Sigma T = 29,600$  lb. (lines 10 to 16, inclusive) L = 66 ft. 1.5 T = 44,400 cL = 19,734 R<sub>D</sub> = 24,666 D = 135,800 = 5.5 in.

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

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

24,600 = 1.7 piles per ft. 12,800

$$D = \frac{12}{1.7} = 6.7$$
 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 road-way 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 slipplane 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 slipplane.

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 (1b )	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 LW 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, 87(					
Normal force (1b )	17, 500	18,200	14, 500	13,750	10,850	8,200	6,300					
	3,920	3,900	3,200	3, 260	2,720	2,410	2,920					

 $\sum N = 89,300 + 154,800 = 244,100 \text{ lb}$ 

 $\Sigma N = 22,330 + 19,900 = 42,230$  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. = 
$$\frac{\Sigma(N - \mu)\tan \phi + cL}{T}$$
 (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 RC	AD GRADE A	T TOP OF S	LIDE						
Increment											
	9- 10	10-11	11-12	12-13	13-14	14-15	15-16	Σ			
Weight of soil (1b )	16,950	15, 550	10,450	9,650	6,950	4, 300	9 50				
Tangential force (lb )	3, 700	3, 270	2,280	2,280	1,690	1,220	40 5	14,845			
Normal force (1b )	16,500	15, 100	10,400	9,550	6,750	4, 180	870	63, 550			
For area between	lines 9 and 16, inc	lusive									
$\Sigma N = 63$	,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  $\phi = 0$  deg., c = 299 lb. per ft.

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

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

= 1.27

Assuming  $\phi = 10$  deg., c = 18 lb. per ft.

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. 1s obtained (data in Table 3):

Assuming  $\phi = 0$  deg., c = 299

S. F. = 
$$\frac{\Sigma N \tan \phi + cL}{ST}$$
 (1)  
= 0 + (299 x 180) = 1.55

Assuming  $\phi = 10 \text{ deg.}$ , c = 18

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

34, 745

= 1.20

Therefore, lowering the grade would fall slightly short of being a permanent solution. The degree of importance to attach to the  $\phi = 10$  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