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AN ABSTRACT OF THE THESIS OF Harry Ludowise for the
Master of Science in Geology presented May 13, 1974.

Title: The Recognition, Investigation, Interpretation
and Treatment of Landslides in the Pacific
Northwest.

APPROVED BY MEMBERS OF THE THESIS COMMITTEE:

[REDACTED]
R. O. VanAtta, Chairman

[REDACTED]
John E. Allen

[REDACTED]
Richard E. Thomas

Certain features unique to the Pacific Northwest influence landsliding in this region. Weakly consolidated sediments, dense vegetation and seasonally concentrated rainfall all contribute to the distinctive conditions that are encountered here.

This study gathers together information applying to the recognition and correction of Pacific Northwest landslides. Treatment practices common to this region are discussed.

A large slide located about 30 miles east of Portland, Oregon along the Clackamas River is used as an example. Exploration, instrumentation, analysis and corrective measures are illustrated.

THE RECOGNITION, INVESTIGATION, INTERPRETATION
AND TREATMENT OF LANDSLIDES IN THE PACIFIC NORTHWEST

by

HARRY LUDOWISE

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requirements for the degree of

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1974

TO THE OFFICE OF GRADUATE STUDIES AND RESEARCH:

The members of the Committee approve the thesis of
Harry Ludowise presented

R. O. Van Atta

John E. Allen

Richard E. Thoms

APPROVED:

Richard E. Thoms, Head, Department of Earth Sciences

David T. Clark, Dean of Graduate Studies and Research

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CHAPTER I

INTRODUCTION

Landslide problems are especially acute in the Pacific Northwest due to the rainy climate. Rainfall concentrated during a few months in the winter causes saturation of the ground, triggering landslides. Pacific Northwest landslides cause several hundred thousand dollars worth of damage annually (Schlicker, 1956). The actual cost is undoubtedly higher than this because much damage is unreported. The Coast Range Mountains are particularly landslide prone due to high rainfall and poorly consolidated sediments. Nearly every major highway cut in these mountains has experienced some slide problems.

I. PURPOSE

The purpose of this study is to prepare a guide to recognition, investigation and treatment of landslides in the Pacific Northwest. This study collects and reports information and data that relate to this area from the large volume of material available on landslides. A local landslide has been utilized as an example of applied investigation and treatment. There are many large slides in this area which are too massive to treat, but it is important to recognize them because frequently a troublesome small slide is merely a reactivation of a portion of a much larger pre-existing slide.

II. DEFINITION AND CLASSIFICATION OF LANDSLIDES

The Highway Research Board, Special Report No. 29, 1958, defines a landslide as follows: "A landslide is the downward and outward movement of slope forming materials, composed of natural rock, soils, artificial fills, or combinations of these materials." Landslide movement may be rapid, as in the case of an avalanche,¹ or the movement may be much slower, taking place over a period of days or even months. The rate of movement in a landslide usually accelerates from zero or a small value and then again slows to a nominal value. In contrast, creep is a slow but relatively constant rate of intermittent movement taking place over a period of years or decades. Creep phenomena and landsliding have gradational boundaries and may be difficult to differentiate in actual practice.

(A number of classification schemes have been proposed by various authors (Terzaghi, 1950; Sharpe, 1938). These classification schemes have been based on the material involved, the shape of the slip surface, or the speed of movement. Terzaghi (1950) has stated, "A phenomenon involving such a multitude of combinations between materials and disturbing agents opens unlimited vistas for the classification enthusiast.") In general, most classification systems have suffered from over complexity so that people in the field never learn enough about a particular slide to properly classify it. Regardless of the difficulties involved, it is still necessary to have some system so that a slide

¹The term avalanche as used in this report refers to the speed of movement and does not imply a composition such as snow.

may be described easily. The classification that will be followed here is one proposed by Varnes, at present the classification most widely accepted (HRB Bull. 29, 1958).

Landslides may be initiated or former slides may be reactivated by human activity. The connection between human works and landsliding may be immediately obvious where earthwork is involved, or it can be a less direct connection where deforestation or the construction of a subdivision has changed the ground water pattern and underground drainage.

(Landslides are frequently studied from two different approaches: that of the geomorphologist who is interested in landsliding as an agent in slope recession and erosion, or from the point of view of the engineering geologist who is interested in the impact of sliding on man's works and methods of minimizing resulting damage.

Actually, landslides can best be studied and described by using both approaches to recognize the causes and effects of sliding. Landslides which are not anticipated or evaluated in advance frequently destroy property which may cost many thousands of dollars as well as endanger human lives.)

III. PACIFIC NORTHWEST SLIDES

There are a number of major slides in the Pacific Northwest which are either large enough or have had sufficient economic impact to have been reported in literature. The Sitkum slide in southwestern Oregon, which resulted in the formation of a lake and caused the deposition of several square miles of lacustrine deposits, is an example (Baldwin, 1964). In addition to the major slides, there are

large numbers of smaller landslides of all types.

(Conditions in the Pacific Northwest that favor landslides are steep terrain, poorly consolidated sediments or sediments with expandable clay content, and high rainfall. The effect of the rainfall is multiplied by the fact that the rain is seasonally concentrated during nine months of the year. These conditions result in a type of rapid moving slide which may be distinctive to the Pacific Northwest. These slides consist of mud containing up to as much as 50 to 60 percent logs and other vegetative debris. These fast moving slides usually are formed in small draws containing intermittent streams. The intermingling of soil and logs remaining after one of these slides is especially difficult to clean up. Because these slides move rapidly, they can be very hazardous and destructive.) Examples of this type of slide include the one which destroyed the U. S. Post Office in Mapleton, Oregon on January 28, 1965. A similar slide closed the road leading to the Crystal Mountain ski area in Washington on July 16, 1971. (Figure 1). These slides are unique because their high debris content seems to help retain the moving material in a compact mass with a high liquid content. The remains of this type of slide can easily be mistaken for a log jam caused by a stream because once motion has stopped, the soil and water drain away leaving only the logs.

IV. ECONOMIC SIGNIFICANCE OF LANDSLIDES

Several hundreds of thousands of dollars are expended annually on slides in the Pacific Northwest (Schlicker, 1956). Many slides are cleaned up and repaired by the persons or agencies immediately involved and no records of the costs are maintained, so the cost is undoubtedly

Figure 1. Crystal Mountain Landslide. Note the high debris content.

larger than that recorded. However, the magnitude of the problem is indicated by the following example. In the Umpqua National Forest, the slides which were large enough to require a contract to correct, cost approximately \$156,000.00 in 1971.² This figure does not include any of the smaller landslides which were cleaned up under timber sale contracts or by maintenance forces, nor does it include slides which occurred on Bureau of Land Management roads and state highways or on private property.

A single slide can vary widely in cost depending on the size. An example of a large slide is one located about one mile north of Port Angeles, Washington on the "Heart O' the Hills Road" in the Olympic National Park which cost approximately \$760,000.00 to correct.² This figure does not include the amount expended on exploration and design.

There are indirect costs involved in landslides, including extra engineering on current construction projects, extra right-of-way costs, disruption of traffic, dislocation of utilities and many other side effects for which no cost accounting is made. Attempts have been made to evaluate the costs of landslides but there are so many factors and agencies involved that they will probably never be accurately measured in terms of dollars. The loss of human life and the interruption of normal commerce would make the cost even greater than a simple addition of damage costs.

²Records Federal Highway Administration, Vancouver, Washington.

CHAPTER II

RECOGNITION OF LANDSLIDES

(Landslides are especially difficult to recognize in the western part of the Pacific Northwest due to the almost universal prevalence of thick vegetation.) In the Coast Range Mountains, the understory vegetation is frequently so thick that one cannot see more than a few feet in any direction, and therefore, the physical phenomena associated with landslides are difficult to identify.

(If an investigation is to be part of a study for engineering work where new construction is to be involved, a preliminary investigation is essential.) This includes a study of aerial photographs and a review of any literature which pertains to the area. In general, there are a few parts of the Pacific Northwest where one government agency or another does not have photographic coverage. The topography, soil types and geology can be evaluated before embarking upon any field work. A good preliminary study will not eliminate any part of the necessary field investigation, but will make it easier to accomplish and more effective.

I. EXISTING DATA

Good sources of literature and information on a particular area of interest include maps and other publications of the United States Geological Survey. State geological surveys also commonly produce maps and publications which prove to be of value in investigating an area for potential landslides. Many of the newer maps may actually show areas

of old landsliding. Utility companies, railroads and highway departments sometimes have unpublished reports which contain useful information. In addition to these sources, soil survey maps and bulletins published by the U. S. Department of Agriculture in cooperation with the states are excellent sources of information. U.S.D.A. soil bulletins published from 1958 to 1961 often contain a section on engineering applications, and after 1961, all the soil bulletins contain a section on the engineering characteristics of soil types.

To be of greatest value in identifying landslides, aerial photographs should be of a scale close to 1 to 10,000. A landslide must be relatively large to show up on a photograph with a scale of 1 to 20,000 or larger. Some of the features that can be discovered on an aerial photograph which help identify a potential landslide are steep slopes, changes in vegetation, the semicircular scar of the head scarp, and disturbed ground with hummocky topography and undrained depressions. Evidence of ground movement also can be detected from aerial photographs such as displacement of a creek to one side of its channel by the encroachment of slide debris into the creek (HRB Bull. 29, 1958).

The majority of landslides which must be corrected are small, less than 100 feet in width and length, and these will barely be distinguishable on a photo with a scale of 1 to 10,000. Because of this, aerial photos may be of the greatest assistance in locating areas prone to sliding rather than for spotting each individual slide. Photos should be consulted whenever available to assure the investigator that the active slide he is working with is not just a small part of a larger mass that is difficult to identify on the ground.

II. RECONNAISSANCE

Landslides can be recognized on the ground by the same features which are identified on aerial photographs. In addition, an on-the-spot reconnaissance will often uncover features which are too small to be identified on an aerial photo, or are masked by heavy vegetation. Many slides are difficult to detect on the ground because of their large size, and close attention to details on the photos is necessary to detect them. One of the more common landslide features which is useful in detecting large slides on the ground is the soil type. Large slides often contain a heterogeneous mixture of boulders, cobbles and fine grained soil, similar to glacial till (Crandell, 1971). Outcrops of rock should be checked and the attitude of the bedding should be compared with the regional attitude. An explanation should be sought if discordant attitudes are found. Landslides can be detected by noting the presence of water-loving vegetation, displacement of linear features, bowed or bent trees, undrained depressions, cracks, scars, and other features normally associated with landsliding such as hummocky ground (HRB Bull. 29, 1958). While none of these features are conclusive in themselves, they furnish a clue that a further investigation on the ground is desirable in search of supporting and confirming evidence for the existence of a landslide. Unless massive earthwork has covered it, there is almost always evidence of pre-existing unstable landslide conditions. Some of the most troublesome and costly slides are the result of reactivation of an older slide mass. In the author's experience, the presence of a number of small slides are indicative of a potential of larger landslide activity.

CHAPTER III

INVESTIGATION OF LANDSLIDES

I. PRELIMINARY INVESTIGATION

The most satisfactory time to recognize a landslide problem is during the preliminary ground reconnaissance which is normally conducted for any significant engineering project. At this stage, plans are usually still tentative in nature and little has been invested in the location. Recognition of a slide problem at this stage can often result in relocation to avoid the problem, or the design can be easily altered to provide for the landslide. The more advanced the planning, the more difficult and costly it becomes to make modifications; therefore early recognition of landslide problems is vital for a safe and economic design of engineering works.

II. EXPLORATION

One of the first steps in the field exploration of a landslide is preparation of a map and cross-sections. The purpose of mapping at an early stage is to provide a working base on which the exploration data can be located and plotted. Maps which are prepared for slide investigation generally contain both aerial and geologic data. One of the first decisions which must be made is the map scale and contour interval. The area to be mapped, the importance and extent of the study as well as the physical topography, will have a bearing on the scale and contour

interval selected for use in mapping. A scale frequently used by engineers is 1" to 100'. This scale will be found useful in a great many slides. If buildings are involved or great detail is to be mapped, a scale of 1" to 10' or 1" to 50' may be the best scale. Slides which cover large areas, such as a square mile or more, may require the scale to be 1" to 500' or 1" to 1000' in order that the entire slide may be included on a single sheet of usable size. The contour interval may be any appropriate to the topography and may range from 2' up to 50'. A suggested contour interval for the average slide is 5' (Zaruba and Mencl, 1969).

A landslide vicinity map should contain references to: property lines, bench marks, stationing on a highway or railroad or other recognizable permanent features; special slide features such as cracks, displacement of fences and any other landslide features; cultural features such as roads, buildings, fences, pipe lines and sewer lines; any possible water sources including streams, rivers, creeks, and ponds; and subsurface investigations such as drill holes, test pits, instrumentation location and the location of geophysical tests.

When exploring a landslide, one of the most important parameters which must be determined is the shape and configuration of the slip plane. Investigators are discovering, with the increased use of inclinometers, that the slip plane is more complex than had previously been believed. In fact, the simple rotational failure may exist only in small slides involving a few hundred cubic yards (Stout, 1971). The chief use of slip plane data is in analysis and correction design.

There are several methods which can be used to determine the depth

and shape of the slip surface. The one most frequently used in the preliminary and reconnaissance stage is inference based on the physical appearance of the slide. This method can be refined by inclusion of actual measurements of the attitudes of the cracks at the head of the slide and the slip surface at the toe. These are plotted on a profile and are then connected with a smooth curve. The result is often a reasonably accurate representation of the slip surface which can be shown on the cross-sections (HRB Bull. 29, 1958).

The combination of inspection and physical measurement may seem to leave a great deal of latitude for personal evaluation, but in actual practice the results are satisfactory and, in most cases, experienced engineering geologists often arrive at almost identical profiles when working on the same slide (HRB Bull. 29, 1958). This method of determining the slip surface has the advantage of being rapid and relatively inexpensive because it utilizes the equipment that most geologists will have with them in the field - the hand level, tape and Brunton compass.

Borings are frequently made to investigate a landslide. When arranging a program of borings, it is best to include as many auger borings as possible so that water-bearing strata information can be obtained along with the rest of the data. The slip plane is usually difficult to detect in borings unless the slip is occurring along a change in soil or lithology. The slip surface is sometimes associated with a water flow, a point helpful in identifying the slip surface in the boring. Continuous sampling is often resorted to, if there is no change in texture or lithology. The slip surface can appear as a thin parting in the soil and is easily overlooked. On rare occasions, the

boring is made large enough to allow a man to descend down the bore hole and actually observe the strata (Leighton, 1966). The major problems with borings are that they are expensive, access is often difficult for equipment, and the slip surface is frequently overlooked in a bore hole.

Geophysical methods, primarily resistivity and seismic refraction are sometimes applied to a landslide in order to determine the slip surface. Earth resistivity is particularly useful in this regard if the slide is recent or still moving. A series of resistivity depth sounding cross-sections are taken at intervals down the length of the slide, normal to the direction of the movement. These tests, when plotted on a profile of the slide, often reveal the depth of the slip surface (Moore, 1965). The earth resistivity technique is very sensitive to changes in resistivity along with depth. This test is advantageous in that the surface area needed to make a test is roughly three times the depth of interest so that it can be conducted on a narrow slide.

Another phenomenon, which is as yet unexplained and is sometimes encountered when conducting resistivity tests on an active landslide, is an electrical disturbance associated with the slip surface. This disturbance is recorded on the resistivity depth test as a sharp displacement of the field curve (Moore, 1965). The electrical disturbance gradually decreases with time after the slide has come to rest. In the author's experience, the tests must be conducted within the month in which the slide movement occurred in order to be successful in detecting this electrical disturbance.

Refraction seismographs have also been utilized in order to detect the slip surface when there is an increase in velocity associated with it. Refraction seismology has the disadvantage that the spread on the surface must be four to ten times the depth of interest. This often results in the line, along which geophones are placed, extending beyond the slide onto non-sliding ground.

Various well logging techniques such as electro logs and gamma logs might also prove valuable in determining the slip surface in a slide. There has been little reference to the use of these methods in literature on the study of landslides. The slip surface frequently contains extra moisture or a change in lithology. Well logging techniques respond to both of these changes so that detection of the slip plane by these methods is a definite possibility.

Inclinometers of various types are frequently installed in active landslides to detect the slip surface. These range from a simple tube down which rods can be lowered in a boring, to instruments costing over \$10,000.00. The use of these inclinometers is limited by two factors: they must be installed in a bore hole and the slide must still be active. The inclinometer measures deformation of a casing which has been placed in the slide. This property is probably the most serious disadvantage because in many cases involving engineering work, the slide must be analyzed and a correction designed before any movement occurs or can be detected. Many slides do continue to make slight movements which can be picked up by a sensitive inclinometer even after the surficial movement has apparently ceased. An additional problem has been that inclinometers usually require the installation of a relatively

large tube that drills commonly available could not install. This problem has been solved recently and several commercially available units will fit inside a 2" diameter tube (Dunnicliff, 1971).

An example of an inexpensive inclinometer is a simple tube, usually of electrical conduit or plastic tubing. Different lengths of rods are lowered inside the tubing and the length of rod just able to pass through the bend in the tube can be related to the degree of curvature of the tube. This system has the advantages that it may be assembled in the field from materials which are available in any hardware store and that few calculations are needed to analyze the data. The disadvantage is that this simple and inexpensive type costs nearly as much to install as the more sophisticated inclinometers, since the primary cost of installation is in boring the hole and backfilling around the tube. In addition, this type has a low order of sensitivity to detect minor movements and is unable to detect multiple slip surfaces. This last problem can be partially overcome by leaving a rod in the bottom of the tube attached by a wire and measuring from both the bottom up and the top down. The Oregon Highway Department has used an improved simple inclinometer, a square tube, which gives better orientation. They have purchased a strain gauge inclinometer which fits the same tubes.

More sensitive and precise inclinometers have been manufactured using various systems including photographic, pneumatic, the Wheatstone bridge, strain gauge and the accelerometer as the principle of operation. The better models of these instruments are capable of detecting deformations in the casing in the order of 1" in 1000' (Dunnicliff, 1971).

Sensitive inclinometers usually have the disadvantage that some calculations must be made before the data can be utilized. Computer programs are available which accomplish these calculations, but if the computer is used, the results will not be immediately available in the field. Most inclinometers also take some time to read in the field since a profile of the entire tube is normally developed. This is accomplished by making readings at some set interval, usually 2'. Some instruments require a positive and negative reading in each direction so four traverses of the tube are required if both the east-west, north-south components of the movement are desired. It normally takes 30 minutes or more to read a tube, depending on the depth of the tube and whether both directions of movements are desired. The results are then reduced and compared with the initial readings which were taken on each tube. This comparison can be done by superposition of the two plots on a light table or the comparison can be made directly in the computer program which reduces the data and prints out the differences between the two sets of readings (Dunnicliff, 1971).

CHAPTER IV

INTERPRETATION OF EXPLORATION AND STABILITY ANALYSIS

I. ANALYSIS DURING EXPLORATION

It is essential that interpretation and analysis be carried on during the entire process of landslide investigation. During the field exploration, new data should be utilized as soon as obtained. It is often advantageous to change the exploration plan from that originally proposed and maximum information from each boring is essential (HRB Bull. 29, 1958).

A vicinity map and cross-sections should be prepared early during the slide investigation processes. All information from drill holes and other sources should be plotted on the map and cross-sections as soon as obtained. Interpretation of the geological information, geophysical results, boring information and laboratory test data is the most demanding part of a landslide study. The success of mathematical solutions and the design of corrective measures depend on the selection of the proper slip plane configuration and the proper soil strength parameters from test data. There have been a number of studies on the selection of strength parameters for stability analysis. Experienced judgment is still of primary importance in the selection of the proper values (Schuster, 1967), (Singh and Lee, 1970).

II. LABORATORY TESTS AND SELECTION OF STRENGTH VALUES

Laboratory testing of cohesive soils has reached a fairly high state of reliability, but landslides are frequently composed of heterogeneous mixtures of soils, and if a number of samples are submitted for testing, a range of values may be received from the laboratory. The type of slide will also have a bearing on the type of laboratory tests run which in turn will have an effect on the values received (Bishop and Henkel, 1964). The formations in the slide will also have an effect on the choice of strength values to be used in analysis. In a fissured clay, for example, the strength of the clay is largely determined by the fissures which are not examined in the small laboratory sample and the laboratory values must be derated by a factor of 5 to 10 (Terzaghi, 1936). The effects of time must be considered in the choice of strengths and the type of analysis performed (Skempton, 1948).

III. STABILITY ANALYSIS

There are a number of methods available to analyze landslides, many of which have been adapted to the use of the electronic computer. Most of the methods of landslide analysis are based on the assumption that the shearing strength of any soil is made up of cohesion and internal friction. The Swedish method of slices was developed by Fellenius (1927) and has remained the basis of most of the analytical methods applied to rotational slides ever since. The method of slices is a technique of breaking a large irregular mass into smaller units.

The forces acting on these smaller units can then be calculated. This method remains the most common method of slide analysis today, even though it results in a slightly erroneous safety factor (Lambe and Whitman, 1969). The reasons for the continued wide use of the method of slices seem to be that it is widely known, simple to use, and the errors are on the safe side. The Bishop method, a modification of the method of slices is considered by many authorities to be the most accurate method of analysing the rotational type of slide (Bishop, 1955). Morgenstein and Price (1965) introduced a method of analysis which enables one to cope with an irregular slip plane. In many slide problems the slip plane can be approximated by a straight line or a series of lines. In this case, an accurate safety factor can be obtained by the wedge method. (Figure 8).

Another method of analyzing rotational types of slides is that developed by Taylor (1937). Students of landslides noted when analyzing rotational slide failures that if the angle of the slope remained constant and the height increased, the failure arc would enlarge in proportion. This being true, a factor could be developed for each angle of slope which would not be dependent on height. Taylor published a set of these factors which he called stability numbers. He also noted while analyzing the large number of slides in checking his data that there were relationships between the center of the most critical arc and his numbers.

Knowing the soil properties and the slope, one can enter his tables and find the most critical arc and the factor of safety. Capper and Cassie (1949) and NAVFAC DM-7 (1971) contain the necessary

tables and directions for their use. This method is especially useful in the design of stable cut slopes in excavation. The tables can be entered with the soil properties and the proposed slope after which one can quickly determine if the proposed slope will be stable.

The following is an example of slide analysis using the Swedish method of slices. No attempt will be made here to go into the background or theory behind this method of analysis.

In order to make a stability analysis of a landslide, it is necessary to have the following information: (1) a definition of the soil mass involved, such as depth and lateral extent, (2) zones of weakness, (3) the strength parameters of the soils involved (weight in pounds γ , cohesion in pounds C , angle of internal friction in degrees ϕ), (4) pore pressure or ground water level, and (5) outside influences which may affect the slide, as irrigation or broken water and storm sewer lines.

Once this information has been assembled, the next stage in making a stability analysis is to construct a cross-section of the slide. The cross-section must be parallel to the direction of movement and at the point of greatest length of the slide. The actual cross-section will be necessarily idealized due to the complexity of most landslides as well as the paucity of information about the slide. If the analysis is to be done by computer, there may be restrictions on the cross-section imposed by the computer program (State of Calif., 1970). If the computations are to be done manually, it will be advantageous if the number of soil layers can be idealized to one layer or two, at the most. Once the cross-section is completed and the soil

properties shown, it will be best to ink the drawing if the problem is to be solved by hand, so that construction lines can be penciled in and erased without destroying the cross section.

In order to save time in both hand calculation and computer time, if one is used, it is necessary to select an initial arc as close to the critical failure arc as possible. In a soil deriving most of its strength from internal friction, a shallow failure arc will have the lowest factor of safety. When a soil derives most of its strength from cohesion, a deep failure arc will usually be the most critical (NAVFAC DM-7, 1971). In some cases, such as that of an embankment with a stabilizing berm, more than one critical arc may exist and it will be desirable to analyze the section using more than one initial failure center so that all potential failures are located.

After laying out the assumed failure arc on the cross-section, the next step is to break up the mass above the failure arc into a series of vertical slices as shown on Figure 2 (Lambe and Whitman, 1969). Six to ten slices is normally considered adequate. They need not be of equal width, but the boundaries should be drawn so that the slices form simple geometric shapes. Slices 2 through 6 can be treated as trapezoids and slices 1 and 7 as triangles. Once the slices have been drawn, and the original arc drawn, the next step is to lay out a table as shown on Figure 2. Different authorities may set up the table in different ways, but the results will be the same (Lambe and Whitman, 1969), (HRB Bull. 216, 1959), (Bishop, 1955).

Slice No.	W	Sin Θ	W Sin Θ	Cos Θ	W Cos Θ	U _i	Δl	$\bar{u}$	$\bar{N}$
1	2.8	.95	2.7	.31	.9	0	12	0	.9
2	5.5	.79	4.3	.62	3.4	.19	6	1.1	2.3
3	6.0	.62	3.7	.79	4.7	.31	5	1.5	3.2
4	7.2	.42	3.0	.91	6.5	.38	5.5	2.1	4.4
5	6.7	.17	1.2	.98	6.6	.34	5	1.7	4.9
6	4.1	.05	.2	.99	4.1	.23	5	1.2	2.9
7	2.1	-.19	-.4	-.98	-2.1	0	6	0	-2.1
				14.7			44.5		16.5

$$F.S. = \frac{C + \sum N \tan \phi}{\sum W \sin \Theta} = \frac{.250 \times 44.5 + 16.5 \times .176}{14.7} = .95$$

Figure 2. Landslide analysis using the method of slices.

The table is filled in as follows: Slice Number - self-explanatory

W - the weight of each slice, found by multiplying the area of the slice by the weight of the soil per cubic foot. This gives the weight of the slice as the slice is assumed to be 1 foot thick.

$\sin \theta$ - the sin of the angle between the horizontal and a line normal to the radius of the failure circle. This angle is usually found graphically from the cross-section.

$W \sin \theta$ - tangential force of the slice; column W multiplied by the sin.

$\cos \theta$ - cos of the angle found for sin.

$W \cos \theta$ - normal force of the slice; column W multiplied by the cos.

U_i - the pore pressure; found by multiplying the depth in feet taken at right angles to the phreatic surface from the bottom of the slice, times the weight of water or 62.4 (Figure 3). This column is not used if water is not present.

l - the length of the failure surface along the bottom of the slice in feet; usually found by picking it off the graph with a scale and assuming the bottom of the slice to be a straight line.

U - the force from water; found by multiplying length by U_i . This column is not used if water is not present.

N - the effective normal force; calculated by subtracting the force from water U from the effective weight of the slice $W \cos \theta$. If pore pressure is unknown, or not a factor, the value of $W \cos \theta$ is used.

Generally the weights found are recorded in thousands of pounds called Kips (K) in order to simplify the note-keeping, but it does not

$$u_i = (6.1)(0.0624) = 3.8 \text{ k}$$

$$U = (3.8)(5.5) = 2.1 \text{ k}$$

Figure 3. Calculation of force from water.

matter what units are used as long as they are used consistently.

The final step is actually calculating the factor of safety for the arc. The factor of safety is defined as follows: the sum of the forces resisting motion divided by the sum of the forces actuating. If the actuating forces are greater than the resisting forces, the factor of safety falls below 1, and the failure of the slope is imminent. The actual equation used to find the factor of safety is:

$$F.S. = \frac{Cl + \sum N \tan \phi}{\sum W \sin \theta} \quad (\text{HRB Bull. 29, 1958}).$$

An example of the actual calculation is shown in Figure 2.

In order to find the minimum factor of safety for a slope, it will be necessary to calculate the factor of safety for several failure arcs. Since the center of rotation may move vertically and horizontally, and the radius may vary, it requires considerable judgment to know when the minimum factor of safety has been found. When there is a firm base to the slide, the circles are all drawn tangent to the base and the factors of safety are plotted as a grid, contours showing equal factors of safety are drawn, and the minimum will then show up as a depression (Lambe and Whitman, 1969). (Figure 4).

IV. STABILITY OF SLIDING SLOPES

If the proper values are chosen for the stability analysis, a safety factor above 1 would indicate that a slope is stable. Most authorities agree that if a sliding slope has a factor of safety slightly less than 1 and with a value of .95 or less, rapid failure can be expected in cohesive soils (HRB Bull. 29, 1958). With this

Figure 4. Minimum factor of safety.

information, it is possible to confirm the stability analysis and choice of soil strength values by reconstructing the original ground surface and finding a factor of safety between 0.95 and 1 (HRB Bull. 29, 1958).

Uncertainty in dealing with unstable slopes is always present due to inconsistencies in the sliding strata, incomplete data resulting in an erroneous cross-section, presence of unsuspected water and other unknown factors. Correct interpretation of the geological and other test data will reduce the uncertainties in slide analysis to a minimum.

CHAPTER V

TREATMENT OF LANDSLIDES

I. PRELIMINARY REMEDIES

Treatment of landslides is expensive and a poorly planned correction can cause more extensive sliding. Therefore, an understanding of the slide must be obtained before any action is recommended. One of the immediate treatments used in the Pacific Northwest is tree removal. This action has been proven to be detrimental and may actually increase the danger of sliding (Gray, 1970). Earthwork particularly should be avoided until after an investigation has been completed. The Pipe Organ slide, which occurred in 1965 near Dillon, Montana, is an example where an attempt to correct a small slide by benching and slope flattening resulted in reactivating an enormous ancient slide (Williams and Armstrong, 1970).

Covering of a small landslide with plastic sheeting is sometimes attempted as a temporary expedient in stabilizing a landslide (Leighton, 1966). This should be considered only if the slope is steep enough to afford immediate runoff of rain water. The plastic prevents water from escaping through evaporation as well as preventing water from entering the soil. If puddles form in depressions on the plastic, water may eventually enter the slide through punctures and increase the moisture problem in the slide.

When a slide is encountered which is endangering property, the best form of immediate action which can be taken is to reduce the amount of water in the area of the slide mass. This action will seldom cause additional sliding problems on the areas adjoining the slide. There are a number of ways that this may be done. One of the most common is to seal ground cracks to prevent the entrance of surface water. This is done by filling them with compacted earth or by pouring powdered bentonite into the cracks. Bentonite, available for use for driller's mud, is used directly as it comes from the bag (NAVFAC, 1971).

Surface water should not be allowed to stand. Ponds and other enclosed depressions should be drained. Drainage can be accomplished by digging ditches by mechanical equipment, hand tools, or the use of explosives. Undrained depressions are very common on old slides and may not have free water standing in them. However, any water which enters the depression will eventually soak into the slide.

When treating slides in built up or urban areas, utilities must be considered. Sewer and water lines can be broken by slide movement, allowing water to escape into the slide. A careful examination must be made because both utilities can continue to function even after being broken (Zaruba and Mencl, 1969).

The most important factor to be kept in mind in planning immediate types of remedies is that until a more detailed examination of the slide is made, nothing should be done which could cause trouble later.

II. PERMANENT TREATMENT OF SLIDES

Treatment of landslides may be classified into three general categories: (1) avoidance, (2) reducing the driving forces and (3) increasing the shearing resistance.

Avoidance

The most obvious answer to landslide problems is simply to avoid them. This may be done by recognizing the hazard in the beginning and, with proper land use procedures, avoiding excavation or construction of anything which can be damaged by slide activity. Avoidance does not necessarily mean relocation in the case of a highway or similar structure. Frequently a small change in grade will accomplish the same effect (HRB Bull. 29, 1958). An example is that of the Southern Pacific Railroad which, in 1902, relocated about three miles of track, abandoning several tunnels and a bridge, after a serious slide located along Cow Creek in Section 12, T32S, R8W, Willamette Meridian.

Another method of avoiding a landslide is to bridge over it. This is most commonly done with linear engineering works such as railroads, highways and aqueducts. Bridging is usually done only when the slide itself is narrow with stable sides on which the bridge can be based. Generally a bridge will be more costly than other treatments and it is only used under conditions where other corrective measures are not practical.

A landslide can also be avoided by removing the slide (NAVFAC,

1971). This is most common when the slide is small. Removal of the slide may also be considered when the value of the endangered work is high or when no failure can be tolerated such as at a dam site with human lives at stake.

Reducing the Driving Forces

Avoidance methods do not have any effect on the stability of the slide. These methods simply prevent the slide from adversely affecting the engineering work or whatever is in the way. Excavation methods attempt to stabilize the slide by reducing the shearing stresses or by increasing the shearing resistance. It is usually more economical to design slopes which will be stable rather than to redesign them after sliding has taken place (Zaruba and Mencl, 1969).

Removal of the head of a slide or slope flattening is based on reducing the driving forces on the slide. When designing this type of slide correction, the entire area must be taken into account and care must be used that the excavation does not remove the support from adjoining areas and cause additional sliding (NAVFAC, 1971).

One of the easiest methods of slide correction is to simply flatten the slope to a safe angle as determined by stability analysis. In fact, there are computer stability programs available which will select the stable slope automatically (State of Calif., 1970). However, there are a number of reasons why this may not be desirable. The topography may be such that slope flattening is impractical due to the high amount of excavation involved. A large flat slope opens a great unbroken surface to rainfall and serious erosion can result (Leighton, 1966).

Benches are usually constructed on slopes which are higher than 20 feet. Hennes, Hawkins and McCoy (1958) found that for a given amount of excavation, slope flattening was less effective in slide stabilization than a slope with benches. Benching has other beneficial effects in that the benches minimize slope wash and, in the case of a nonhomogeneous soil, provide a means for equipment to get up on the slope to clean up small failures and generally maintain the slope.

Slope flattening will be found to be the most effective on the rotational type of slide and is generally not recommended for the mud flows or slides which have straight slip planes (HRB Bull. 29, 1958). As a rule of thumb, if a slide was caused by excavation at the toe, it will be necessary to remove approximately twice as much material at the head of the slide to stabilize it (HRB Bull. 29, 1958). In many cases, slope flattening with or without benches is used in connection with other slide correction techniques. The excess excavation material resulting from slope flattening can result in a serious waste disposal problem. Utilizing the excess to construct a counterbalance at the toe of the slide is one excellent solution to the problems of slide stabilization and waste disposal.

Increasing Shearing Resistance

Another means of slide stabilization through excavation methods is the shear key, which consists of a wedge of higher strength soil or rock placed near the center of the sliding mass. (Figure 6). The shear key extends down through the slip plane into the firm underlying soil or rock (NAVFAC, 1971). The size of the shear key and the type of soils necessary for its construction can be determined through

ordinary landslide analysis methods. The disadvantages in using a shear key type of slide correction are the difficulty in locating a nearby source of high strength soils and the problems involved in the necessary excavation. The latter is probably the most serious objection since removing soil to an elevation below the slip plane in the middle of a landslide can cause further instability problems. Safety is of utmost importance in the design of this type of slide correction. Generally a shear key slide correction will only be found economically feasible in a rather large slide where the thickness of the sliding mass is thin compared to the length (Leighton, 1966).

Drainage is particularly effective in slide stabilization as it both increases the shear strength of the soil and reduces the driving forces acting on the slide. The primary effect of drainage on a slide is the reduction of the pore pressure affecting the slide (Hennes, Hawkins and McCoy, 1958). Water can be removed from the slide by means of both surface and underground drainage methods. One of the first steps in the prevention and treatment of slides is to obtain good surface drainage. If possible, all streams and temporary water courses should be diverted from the slide and any standing water, both on the slide and near it, must be drained. Sometimes paved ditches and culvert pipes are utilized in order to keep collected water from escaping back into the slide. However, these are subject to rupture if the slide continues to move (Zaruba and Mencl, 1969). It will usually be found that if the topography lends itself, open ditches are a very effective way of draining the slide. Once water is collected at the surface and placed in a ditch and kept flowing, the bottom of

the ditch tends to seal itself with silt and very little water will escape back into the slide. During drainage operations, the existing ground cover should not be disturbed any more than necessary since vegetation removes water from the ground through transpiration of the plants (Gray, 1970).

One of the important features which must be determined during the preliminary investigation is the presence of underground water. The drill log should contain the elevation of the top and bottom of the water bearing stratum. Subsurface drainage can consist of both drilled horizontal drains and ordinary drains placed during construction of an embankment. Drains placed under an embankment usually consist of a layer of free-draining gravel and a series of perforated culvert pipe to allow escape of any water picked up by the gravel. Occasionally a layer of rock is placed under a fill without using any pipe and the water escapes through the voids in the rock (NAVFAC, 1971). This is the so-called "French drain." (Figure 5). This type of drainage is only practical under new construction or in the course of major slide reconstruction.

If the water is encountered in a slide at a shallow depth, a cutoff drain may be designed (HRB Bull. 29, 1958). (Figure 5). This consists of a deep ditch dug to the bottom elevation of the water bearing stratum and across the slide. The cutoff trench will have a perforated pipe placed in it and be partially filled with free-draining gravel. Ten to fifteen feet is the normal economic depth of a cutoff drain.

Numerous methods have been used to drain landslides if the water

Figure 5: Drainage methods.

level is too deep to be reached from the surface. Drainage tunnels have been used on some slides in Oregon, but are seldom used today due to high labor costs (Anon, 1947).

Another system of underground drainage which is sometimes used, if the geologic conditions are suitable, is the vertical drainage well. This is used when the water affecting the slide is perched and there is an aquifer below which is capable of accepting water from above (NAVFAC, 1971). When this method is used, care must be taken that the water table in the lower aquifer does not rise during periods of high water, thus introducing water back into the landslide.

Drilled in horizontal drains have supplemented most other means of underground drainage methods in recent years. (Figure 5). This is due to the low cost and the fact that the drain installation causes little disruption to the landscape. In 1974, drilled in horizontal drains cost between \$5.00 and \$5.50 per foot, including the pipe.³ The cost of stabilizing a slide using horizontal drains usually works out to be 5 to 10 percent of the cost of slide stabilization using other means. Horizontal drains have the advantage that they can be installed very rapidly. A crew of two with a drill designed to install horizontal drains can install approximately 600 feet of drain per 8 hour shift in good drilling conditions.³ Originally the drain pipe was modified water pipe with holes drilled in or burned in with an acetylene torch. Later perforated galvanized pipe became available commercially. At present, nearly all the horizontal drain pipe is constructed from PVC plastic which can be perforated or slotted. The

³Records, Federal Highway Administration, Vancouver, Washington.

slotted pipe is rapidly becoming the most popular since the width of the slots are designed to be of optimum size for capillary attraction and also function as a well screen. Horizontal drain pipe is commonly 2 inches in diameter with 3 or 4 inch pipe being used on occasion. Horizontal drains are generally limited to a depth of 200 to 300 feet. However, they have been drilled as deep as 1500 feet (HRB Bull. 29, 1958). There is no mechanical limit to the depth that drains can be drilled, but after 300 feet one loses a measure of directional control and the drill will sometimes break out through the surface of the ground, particularly if the material being drilled is bouldery. This being the case, one can well imagine how many deep holes go astray without the operators being aware of the deviations.

Because horizontal drains require a gradient to operate, long drains may be ineffective because of poor directional control. When a slide is so large that long drains are required, such drains are frequently brought to a central spot in the slide, with drainage originating from there. This is usually accomplished by digging an excavation wide and deep enough to lower a drill to the desired elevation. Then a series of drains are drilled out of this excavation. The State of Washington has drained these excavations by self-starting siphons or by drilling another drain into the pit (HRB Bull. 29, 1958).

Horizontal drains are normally drilled with a gradient of 5 to 25 percent, the angle being determined by the respective elevations of the drill site and the base of the wet stratum. Spacing between drilled in horizontal drains is sometimes determined by field judgment

after a fan of drains has been placed across the slide. The drain showing the greatest flow will be bracketed by a drain on either side. The spacing of horizontal drains can also be determined by a rational method once the permeability of the slide is known (Todd, 1959), (Toksoz and Kirkham, 1961).

Because reduction of the pore pressure in the slide has a great stabilizing influence on a slide, observation wells or piezometers should be installed in the preliminary exploration holes. The effect of the drain system can then be evaluated as the drains are installed. This evaluation is particularly important since it is possible to get a reasonable flow of water from a drain installation and yet find that the hydrostatic head has not been lowered (HRB Bull. 29, 1958). Occasionally a small flow will prove effective in reducing the head in the slide, but if the installation were evaluated on the basis of the flow, it would be judged a failure. Observation wells also allow the continued monitoring of the slide since drains require periodic maintenance. A program of checking observation wells can provide advance warning that whatever stabilization technique was used may be in imminent danger of failing.

Restraining structures can be constructed of materials other than rock or earth, but these are exceptions used infrequently due to the size of the forces involved in landslides. Piling, cribs, dowels, rock bolts and tie rods have all been used as restraints (HRB Bull. 29, 1958).

Slides which are predominately in hard bedrock and are moving along fractures have been successfully restrained by rock bolts. Rock

bolts are a special case and do not depend directly on the shear strength of the steel for their restraining effect but on the friction generated as the rock is forced together under compression. The design of successful slide correction, using rock bolts, requires considerable exploration. Since the rock bolts work by pulling the rock into compression, the minimum depth to which the rock bolt must be anchored is critical (NAVFAC, 1971).

A notable case where piling have been used successfully to restrain landslides is in Seattle, Washington along Interstate Highway 5. Here a number of large slides were restrained by a combination of drainage and large cast-in-place cylinder piles. (Figure 6). Correction by the use of piling is not always successful as the forces involved are too great to be arrested by piles, or the sliding material will flow through the intervals between the piles. Piling, if unsuccessful, can have a negative effect on the slide if they are moved by the action of the slide. The material into which they have been driven will be disturbed and the slide can deepen. As a general rule, for pilings to be successful they must penetrate $1/4$ to $1/3$ of their length into stable material and there should be at least one pile for every 50 cubic yards of moving mass (HRB Bull. 29, 1958).

Cribs, retaining walls and bulkheads are most likely to be effective if installed before sliding occurs (HRB Bull. 29, 1958). Once a landslide moves, it disrupts the soil and some of the strength of the soil mass is lost because the remolded strength of a clay is less than the original strength (Lambe and Whitman, 1969). These structures must resist both the shearing action of the slide and the overturning

action. The most critical factor in designing this type of landslide correction is to provide for drainage through the structure.

Buttresses are probably the most common restraining structure which are designed to correct slides. (Figure 6). These are constructed at the toe of the slide and are usually made of earth or rock. Buttresses and counterbalances are usually constructed of a material stronger than the native soil. This is not necessary in the case of the counterbalance design where only the mass of the material is utilized. Counterbalances are constructed at the toe of a slide where the slide material forms an upward bulge. The design is critical in that if weak material from the slide is used to construct the counterbalance, it must guard against the instability of the counterbalance itself. A combination of slope flattening and buttressing is often found to be a most effective means of slide correction (NAVFAC, 1971).

The design of restraining structures follows the normal soil mechanics method of slide analysis. There are computer programs available which will automatically analyze the slide and construct the counterbalance to a preselected factor (State of Calif., 1970). Caution must be used here because the counterbalance can automatically be constructed high enough to have lateral failures. Generally, a stability analysis is made on the center of the slide so when designing a buttress, it is advisable to check a section on either side of the center as well. In addition, one should be sure that the slopes used in the buttress or counterbalance will be stable. A second situation which commonly occurs when a counterbalance is used is that there will be two critical failure arcs. One will be a large radius deep arc

which will involve the whole slide and counterbalance. A second will be less deep, cutting the surface of the ground just above the buttress or counterbalance (Zaruba and Mencl, 1969).

There have been two general methods used to stabilize landslides by hardening the soils - chemical and electrical.

Chemical methods generally consist of the addition of a chemical such as lime, calcium, chloride or phosphoric acid. These will react with the clay minerals present in the soil, changing them to a less sensitive clay or reducing their affinity for water (NAVFAC, 1971). This technique has been used by the Oklahoma Highway Department on several locations.⁴ Briefly, their technique is to make borings to the depth of the slip surface at an approximate spacing of 10 feet. These borings are then filled with lime. They feel that for this method to be effective, the slide must be made up of A-6 to A-7 soils with the moisture content slightly above optimum. Evaluation of the slides treated in this manner indicates that the major effect has been water removal and lowering of the optimum moisture of these soils rather than increasing the strength of the soils.

A recently developed proprietary process also stabilizes slides using chemical means.⁵ The method is based on the introduction of one or more chemicals which react with the existing soil minerals causing a permanent change. The actual chemicals used are a trade secret and are selected on the basis of laboratory tests. This method does not

⁴Personal communication, Charles V. Robertson, Materials Engineer, Oklahoma Highway Department.

⁵Ion Tech, Inc., 1680 Bryant Street, Daly City, California

require that the slide be primarily clay, and it can be composed of a large percentage of silt. It does require that the existing slide have a safety factor of approximately 0.95.

The chemical processes are relatively inexpensive compared to other methods of landslide correction.

Another method of hardening soils is the electrical method. Electrical pressure can be used to remove water from the earth - a process defined by the term electro-osmosis. This has been applied to horizontal drains to make them function more effectively. Casagrande (1953) recommends the use of electricity for simultaneous soil hardening and water removal and holds U. S. patents on the process. The principle is that the water well or horizontal drain pipe acts as the cathode. The anode is usually aluminum. The electrical current ionizes the aluminum which then exchanges with sodium or calcium ions in the clay causing a permanent change in the clay.

The Casagrande method was used in King County, Washington to stabilize a sliding condition on the east end of the Evergreen Point Floating Bridge across Lake Washington in 1962.⁶ Power was furnished by two 10KW welding generators with 140 cathodes and 70 aluminum anodes. The dewater hardening processes operated 24 hours a day for six days. 20 to 25 gallons of water were removed per day from each cathode and a solidified column 16 feet in diameter was formed around each anode.

A number of other methods of slide stabilization have been tried at various times. The only two which are in use at present are

⁶Records , Federal Highway Administration, Olympia, Washington

blasting and rock blankets. Blasting is seldom used today. The method consists of increasing the shear resistance by disrupting the slip plane and fracturing the sliding surface to allow water to escape (Zaruba and Mencl, 1969).

Rock blankets are used extensively on small mudflow slides in the Pacific Northwest with considerable success. The principle of the method is to place a covering of free draining material on the unstable slope. Either rock or gravel may be used, although tradition favors large shot rock. The rock blanket, usually 3 to 5 feet thick, is placed on the slope of a mudflow type slide. This porous material allows the water to escape while restraining the soil. Rock blankets are designed on the basis of practical experience (NAVFAC, 1971).

CHAPTER VI

CLACKAMAS LANDSLIDE

I. INTRODUCTION

The following is an example of analysis and treatment of a Pacific Northwest landslide. This slide is located about 30 miles east of Portland, Oregon along the Clackamas River in the NE $\frac{1}{4}$ Sec. 19, T4S, R6E, Willamette Meridian. Excavation of a cut during reconstruction of the Clackamas Highway in 1968 triggered a slope failure which blocked the highway and endangered a key utility tower. The initial failure took place rapidly, during a period of heavy rainfall, and was witnessed by a construction worker. According to this witness, a large mass of earth about 15 feet high moved rapidly across the highway. The majority of this material disappeared into the adjoining river. After this slide movement was over, the only visible evidence remaining was the 3 to 6 feet of debris on the road, the scar on the slope and some muddy brush caught on the telephone line located between the road and the river.

Because this slide posed a serious threat to the high voltage transmission tower as well as to the highway, an extensive exploration and analysis program was undertaken on this slide by the Federal Highway Administration.

II. BACKGROUND

The preparation of a contract to reconstruct portions of the Clackamas Highway, State Highway 224, were nearly complete in December 1964 when the road suffered serious damage due to flooding. After emergency repairs were completed to make the highway passable to traffic, intensive redesign work was undertaken to add the flood damage repair to the reconstruction contract. The federal funds available for flood damage repair had to be expended in a limited time making rapid completion of the redesign work imperative. As a result of the need for haste, a very limited soils survey was done on the flood damaged areas and this slide was not identified during the design stage.

The terrain in the vicinity of the slide is comprised of nearly vertical basalt cliffs between moderately rolling uplands and the Clackamas River. Bedrock, where exposed is mapped as Boring agglomerates (Baldwin, 1964).

The nearest U. S. Department of Commerce Weather Bureau Station is at the Three Lynx Ranger Station. Using data available from this station, the slide vicinity is found to have average annual precipitation of 65.5 inches including a mean annual snowfall of 34 inches. The mean temperature is 51 F, but extremes may range from 105 F to a low of minus 4 F.

The original back slope was designed on a .75:1 which proved too steep for stability. In addition to the large failure described earlier, a series of smaller failures took place during construction.

As the project was under construction, it was decided that a delay for investigation of the slide could not be tolerated. It was arbitrarily decided to correct the slide by flattening the slope to 1.5:1. This did not prove effective and additional small failures occurred during the remainder of 1969.

It was then decided that a complete investigation should be made of the slide. This decision was made primarily because Portland General Electric's high voltage transmission tower 97 appeared to be in danger if sliding continued. This tower is especially critical as it is the highest in a series of towers. If it should prove necessary to move tower 97, as many as 10 other towers could require movement or modifications.

III. EXPLORATION

In order to accomplish the field exploration in the most efficient manner, aerial photographs, plans, cross-sections and soils survey data were reviewed. The aerial photographs revealed that the slide was only a part of more extensive sliding which extended to the east and west along the Clackamas River. A large slide scar was found just west of the slide under study. (Figure 7).

The detailed field investigation took place during December 1969. A program of mapping, drilling, sampling, geophysical (seismic) testing and instrumentation was implemented. (Plate 2). A truck-mounted Acker core drill was used with both auger and regular drilling equipment to make the exploratory borings and obtain the desired samples. A total of six borings were made in the slide area. Because the slide

1 Mile

Figure 7. Station 425 Clackamas Highway slide.

activity had been related to periods of heavy rainfall, as much of the boring as possible was accomplished using augers which do not introduce water into the ground while drilling. Augering was made difficult by the presence of boulders in the slide material. Only one of the six borings was completed using augers alone. The others were augered to refusal and the boring completed using a 3-3/4 inch tricone rock bit. The water table was measured immediately in the augered hole and again after a several day period to allow water to reach equilibrium in the rock bitted holes. In order to provide for future water table measurements, one inch electrical conduit was placed in five of the six holes for use as observation wells. Since the safety of the Portland General Electric transmission tower was of primary consideration during this investigation, an inclinometer tube was placed in boring number 3 near the tower.

Sampling of the different soil strata was done by means of a shelby tube sampler and a stationery piston tube sampler. Auger cuttings were also collected and sealed in glass jars. Samples of the upper slide debris were collected with little difficulty, but considerable difficulty was experienced in sampling the wet soil associated with the slip plane. The difficulty was primarily that the layer was thin and underlain by a harder stratum which shelby tubes could not penetrate. In several cases the layer was not detected until after the layer had been completely penetrated. This was especially true when the rock bit was being used. The use of water when rock bitting through large boulders softened the soil, making it impossible to distinguish between the naturally occurring hard and soft layers.

Water introduced during drilling also made it difficult to identify the water bearing strata, if present.

Five refraction seismic lines were run on this slide using an Electro-Tech PRA-2 12 channel seismograph. The lines were tied to the borings in an attempt to expand the information economically on the slip plane obtained in the borings. Line 5 was run halfway up the slope providing slip plane data in a location where the information was extremely useful and would have been difficult to obtain by other means.

The Electro-Tech instrument provided a permanent record, although the Kodak 1875 self-developing paper which was used resulted in a record unsuitable for reproduction. The data was reduced using an IBM 360 electronic computer with the resulting interpretation spot-checked by hand calculations. Each seismic line was shot in the center of the line in addition to the two ends in order to increase the number of end-point determinations. The seismic data showed a three layer condition with 1500 FPS and 3500 FPS layers above the slip plane and 6000 FPS in the firmer material below. (Plate 3)

The field investigation revealed that the moving slide material consisted of soil boulder slide debris. This material was sliding on a thin layer of reddish brown medium plastic moist clay. The material under the slip plane was a firm weathered rock which in turn overlay a harder rock. One of the shelby tube samples contained a six inch layer of very porous volcanic tephra recovered from a layer just above the weathered rock layer. There may be a discontinuous layer of this porous material in the slide which is conveying large amounts of water

to the slip plane.

IV. LABORATORY TESTING

The laboratory testing consisted of routine identification and one unconfined compression test. (Table 1). The number of laboratory tests was severely limited because of the poor sample recovery in the field. Some undisturbed samples recovered in shelly tubes contained trapped water. The samples flowed out of the tube when opened in the laboratory so that valid tests could not be performed on this material.

The Atterberg limits, moisture content and grain size distribution were used to estimate the strength of the cohesive material.

V. STABILITY ANALYSIS

The landslide at station 425 appears to have had two and possibly more active stages in its history. The exploration indicates a nearly flat surface associated with the top of the weathered rock. The current movement is largely rotational with some mudflows taking place near the toe. Because of this, two methods of stability analysis were used in analyzing this slide.

Wedge analysis (Figure 8) was used to analyze the sliding type of failure associated with the earlier sliding movement. The wedge analysis shows a factor of safety well above unity, tending to confirm the field observations that the current sliding is of a rotational nature. These results must be applied with some caution since the soil strength data on which the calculations are based was limited due to the poor sample recovery at the slide. It is probable that the

TABLE I

LABORATORY DATA CLACKAMAS LANDSLIDE

Boring No. 1, Location Station 324+50, 240' Lt. $\angle$

Sample No.	1	2	3
Depth	20 Ft.	24.5 - 26.5 Ft.	34 Ft.
Sieve Size			
Percent Passing			
No. 4			100.0
No. 10			95.3
No. 40			83.6
No. 200			62.6
0.02 MM			47.5
0.005 MM			30.5
Natural Moisture	41.0		40.5
Specific Gravity			2.74
Liquid Limit			45
Plasticity Index			16
Shearing Stress		0.47 Tons/Foot ² 75.5 Lbs./Foot ³ at 52.0%

$$\frac{112 \times 56}{2} \times 114 = 358K$$

$$\frac{374 \times 36}{2} \times 114 = 350K$$

$$358 + 350 = 708K$$

$$W_n = W \cos \theta \quad 708 \times .951 = 673$$

$$W = W \sin \theta \quad 708 \times .309 = 218$$

$$S = W_n \tan \phi + CL \quad 673.0 + 940.374 = 352$$

$$Pa = \frac{\gamma H^2}{2} - 2CH \quad \frac{114(56)^2}{2} - 2(940)(56) = 74$$

$$Pat = Pa \cos \theta \quad 74 \times .951 = 70$$

$$SF = \frac{S}{Pat + Wt} \quad \frac{352}{70 + 218} = 1.22$$

Figure 8. Wedge analysis.

natural slope was close to equilibrium having been formed by sliding originally. This fact was utilized to confirm the soil strength values used by performing an analysis using the original slope and finding a safety factor of 1.

The slide was also analyzed using both the ordinary method of slices and the modified Bishop method on an IBM 360 electronic computer. These results show a factor of safety of less than 1 for a failure arc just touching the top of the weathered layer. Figure 9 shows an example of one of the computer analysis.

A number of analysis, both wedge and circular, were prepared designing a buttress to correct this slide. It was found that the most economical buttress which could be designed would consist of rock having weight of 140 pounds per cubic foot and an angle of internal friction of 40 , and in the shape of a parallelogram 40 feet thick and 30 feet high. Analysis also showed that a reduction of the water table would have a stabilizing effect on the slide.

VI. CORRECTION

The stability analysis indicated that the head of the slide had only marginal stability and that additional slope flattening would not be advisable. An acceptable factor of safety could be obtained by constructing the rock buttress previously described, but it would be an extremely costly solution due to the size and the need for the construction of the base to be about 10 feet above the highway grade. The excavation necessary at the toe of the slide would also involve some risk of failure during construction before the buttress was in place.

01 MT 01 02 ** S L O P E S T A B I L I T Y A N A L Y S I S **
CLACKAMAS HWY SLIDE STA 425

PAGE 2

GRID PATTERN / 30 SLICES / 10 HOR. PTS. / 5 VERT. PTS. / 40. FT. SPACING / 10. FT. ARC INCR.

LN OR PT NO	END END		SECTION DESCRIPTION		UNIT	COHES ION	TAN OF PHI	PORE PRESSURE	SLOPE OF LINE
	A	B	-X- COORD	-Y- COORD					
1	1	2	494.	891.	114.	940.	0.0	0.0	-0.072
2	2	3	411.	897.	114.	940.	0.0	0.0	0.483
3	3	4	293.	840.	114.	940.	0.0	0.0	0.391
4	4	5	206.	806.	114.	940.	0.0	0.0	0.671
5	5	6	66.	712.	0.	0.	0.0	0.0	0.216
6	6	7	494.	838.	0.	0.	0.0	0.0	0.013
7	7	8	420.	822.	0.	0.	0.0	0.0	0.529
8	8	9	270.	820.	114.	500.	0.176	0.0	0.216
9	9	10	494.	837.	114.	940.	0.0	0.0	0.013
10	10	11	420.	821.	114.	940.	0.0	0.0	0.529
11	11	12	270.	819.	114.	500.	0.176	0.0	0.210
12	12	13	66.	711.	114.	500.	0.176	0.0	0.375
13	13	14	282.	797.	140.	2000.	0.839	0.0	0.226
14	14	15	494.	836.	140.	2000.	0.839	0.0	0.361
15	15	16	282.	788.	0.	0.	0.0	0.0	0.0
16	16	17	66.	710.	0.	0.	0.0	0.0	0.0
17	17	18	495.	640.	0.	0.	0.0	0.0	0.0
18	18	19	65.	640.	0.	0.	0.0	0.0	0.0
19	19	20	80.	840.	0.	0.	0.0	0.0	0.0

SAFETY FACTOR	-X- COORD	-Y- COORD	CRITCL RADIUS	DRIVE (TONS)	RESIST (TONS)	NO OF RADI
*****	80.	840.	1.	*****	*****	0
1.054	120.	840.	102.	63.82	67.29	4
1.015	160.	840.	87.	67.07	68.08	10
1.214	200.	840.	74.	60.62	73.57	15
1.675	240.	840.	58.	38.73	64.87	15
2.018	280.	840.	43.	26.49	53.45	18
2.035	320.	840.	37.	24.86	50.58	15
5.389	360.	840.	137.	275.95	1487.08	7
9.789	400.	840.	88.	91.21	892.79	4
9.742	440.	840.	73.	66.50	647.84	1
1.526	480.	840.	156.	77.94	118.96	2
1.130	520.	840.	133.	61.04	69.00	6
0.904	560.	840.	127.	96.04	86.79	10
0.938	600.	840.	114.	92.07	85.34	14
1.347	640.	840.	96.	67.93	94.18	17
1.234	680.	840.	92.	66.97	82.78	17
1.034	720.	840.	78.	85.27	88.18	12
1.129	760.	840.	63.	64.06	72.34	8
1.935	800.	840.	60.	37.82	73.20	7
27.184	840.	840.	43.	7.32	63.14	2
1.043	880.	920.	189.	79.04	82.43	3
1.002	920.	920.	172.	92.25	82.41	7
0.906	960.	920.	162.	103.62	93.90	10
1.058	1000.	920.	144.	86.68	91.73	13
0.993	1040.	920.	135.	103.99	103.22	16
0.978	1080.	920.	121.	105.16	102.81	13
0.821	1120.	920.	114.	123.34	101.29	9
0.945	1160.	920.	105.	102.25	96.60	8
1.612	1200.	920.	95.	57.51	92.71	7
18.573	1240.	920.	47.	2.27	42.13	2
0.917	1280.	960.	229.	104.11	95.42	5
0.839	1320.	960.	216.	127.17	106.75	7
0.873	1360.	960.	200.	123.46	107.83	20
1.009	1400.	960.	181.	103.45	104.40	13
0.924	1440.	960.	171.	124.88	115.43	13
0.831	1480.	960.	160.	139.52	115.89	10
0.789	1520.	960.	140.	104.82	82.74	7
0.925	1560.	960.	141.	107.64	99.55	6
2.769	1600.	960.	113.	37.55	77.68	5
29.320	1640.	960.	79.	1.34	39.44	1
0.955	1680.	1000.	261.	96.48	95.99	5
0.818	1720.	1000.	252.	135.91	111.21	8
0.827	1760.	1000.	239.	150.80	124.72	10
0.822	1800.	1000.	225.	158.04	129.86	12
0.850	1840.	1000.	208.	146.81	124.70	10
0.749	1880.	1000.	200.	162.04	121.29	8
0.793	1920.	1000.	187.	137.05	108.64	6
1.739	1960.	1000.	164.	74.69	92.07	5
2.655	2000.	1000.	143.	27.22	72.27	4
48.206	2040.	1000.	117.	0.36	41.49	1

Figure 9. Computer analysis Clackamas Highway.

For these reasons, correction using a rock buttress was abandoned in favor of other corrective measures.

Reduction of the water pore pressure had also been shown by stability analysis to be effective in stabilizing the slide. A system of drainage utilizing both surface ditches and horizontal drains was proposed as the most economical method of correcting the slide. Water entered the auger holes rapidly during the exploratory drilling indicating that drains should be effective since the formation was reasonably permeable and water was present.

Approval was obtained for an attempt to correct this slide by drainage. A drainage ditch was surveyed and staked out to drain the swamp shown on Plate 2. Permission could not be obtained to do the necessary dozer work to dig the drainage ditch or to site the drill for installing horizontal drains in the most favorable location. Permission was obtained to install the drains if the existing cut slope was not disturbed and a less desirable drain site was selected at a higher position on the slide.

In March 1972 fifteen drilled in horizontal drains and three additional exploratory borings numbered 7, 8 and 9 were installed in the slide. The drains were planned as a first step in stabilizing the slide to see if they would be effective in reducing the water levels in the slide.

The results do not appear to be completely successful since, although the drains are working and run varying amounts of water from 0 during the dry summer up to 4 gallons per minute measured in the winter, the water table as measured in the test borings still remains

at approximately the same elevations as before the installation of the drains. The fact that the drains were not successful was proven when an additional failure occurred in January 1974 after a period of heavy rains. This most recent activity occurred along the scarp previously formed and shown on the map. (Plate 2). Movement along this scarp is estimated to be about one foot. About 500 cubic yards of debris was deposited on the road in the spring of 1974.

VII. CONCLUSIONS

This slide was not recognized before construction started due to the lack of time for a normal soils investigation. Had this not been a rush project to repair the flood damage, the slide would undoubtedly have been identified with the use of aerial photographs and the other techniques described earlier as part of the normal soils survey. The topography in the slide area and the design requirements for grade plus the alignment of the highway would have made it impossible to avoid the slide in any event. It certainly would have proven more economical to include the slope flattening in the original contract, had the slide been recognized, than doing it later by change order during construction as was done.

Corrective measures were taken in the field before any exploration was accomplished on the slide which is risking the possibility of making a bad situation worse. In this case, the subsequent investigation showed that slope flattening did contribute to the stability of the slide.

The information gathered indicates that this slide is a reacti-

vation of a small portion of a large ancient slide caused by steepening a slope in the process of road construction. The original slide appeared to be a block glide. The current movement is rotational with some mudflow activity occurring at the toe.

The inclinometer installed near tower 97 shows that movement has not taken place since its installation on January 5, 1970 to the most recent reading on March 15, 1974.

The horizontal drain installation, while removing water from the slide, was not successful in lowering the water table in the slide vicinity.

Recommended future corrective measures include flattening the slope to a minimum of 2 to 1 and the addition of a rock blanket to prevent mudflows. Additional drilled in drains should be installed lower on the slope as a further effort to lower the water level in the slide.

Careful monitoring of all the instrumentation will be needed for several years to provide a warning should the head of the slide expand and endanger the power line tower.

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TYPE OF MOVEMENT	TYPE OF MATERIAL (BEFORE MOVEMENT)*	
	BEDROCK	SOILS (CLASTIC MATERIAL, INCLUDING ROCK FRAGMENTS, SHEARED BEDROCK, ORGANIC MATTER, ETC.)
I. FALLS Mass in motion travels most of the distance through the air. Includes free fall, movement by leaps and bounds, and rolling of rock and debris fragments without much interaction of one fragment with another.	ROCKFALL extremely rapid Plate I-a	SOILFALL very rapid Plate I-b
II. SLIDES Movement caused by finite shear failure along one or several surfaces which are visible or whose presence may reasonably be inferred. A. Material in motion not greatly deformed. Moving mass consists of one or a few units. Maximum dimension of units is greater than displacement between units. Movement may be controlled by surfaces of weakness such as faults, bedding planes or joints. Backward tilting of units is common. 1) SLUMP: Movement only along internal slip surfaces, which are usually concave upward. 2) BLOCK GLIDE: Movement of a single unit out and down along a more or less planar surface of weakness, generally a bedding plane. Block may glide far out on original ground surface. B. Material in motion is greatly deformed or consists of many semi-independent units. Movement frequently is structurally controlled by surfaces of weakness such as faults, joints, bedding planes, variations in shear strength between layers of bedded deposits, or by the contact between firm bedrock and overlying detritus. Maximum dimension of units is comparable to or less than displacement between units, and generally much smaller than displacement of center of gravity of the whole mass. Movement may progress beyond original slip surface so that parts of mass slide over the ground surface.	ROTATIONAL Plate I-c SLUMP extremely slow to moderate	PLANAR Plate I-g BLOCK GLIDE slow
	PLANAR Plate I-d BLOCK GLIDE moderate Plate I-i ROCKSLIDE very slow to extremely rapid	ROTATIONAL Plate I-h SLUMP (EARTHFLOW) very rapid Plate I-k FAILURE BY LATERAL SPREADING very rapid
III. FLOWS Movement within displaced mass such that the form taken by moving material or the apparent distribution of velocities and displacements resemble those of viscous fluids. Slip surfaces within moving material are usually not visible or are short-lived. Boundary between moving mass and material in place may be sharp or a zone of distributed shear.	ALL UNCONSOLIDATED	
	MOSTLY LARGE ROCK FRAGMENTS Plate I-l extremely rapid ROCK FRAGMENT FLOW (Variety: ROCKFALL AVALANCHE) This type of movement occurs only when large rockfalls and rockslides attain unusual velocity. E. Elm, Switzerland (1981) after Heim (1932). 1.8 km	MIXED ROCKS, SOIL, CLAY, ETC. Plate I-m SAND RUN rapid to very rapid Plate I-n LOESS FLOW (dry) (caused by earthquake) extremely rapid gradational series Plate I-o DEBRIS AVALANCHE very rapid to extremely rapid Plate I-q very rapid RAPID EARTHFLOW MUDFLOW Plate I-r DEBRIS FLOW very rapid
IV. COMPLEX LANDSLIDES Movement is by a combination of one or more of the three principal types of movement described above. Many landslides are complex, although, as illustrated in Pls. I-k and I-l, one type of movement generally dominates over the others at certain areas within a slide or at a particular time in the evolution of a slide.	MOSTLY PLASTIC Plate I-p SLOW EARTHFLOW slow to rapid MUDFLOW
	WET GRADATIONAL WATER CONTENT DRY	

Nomenclature of the parts of a landslide
(see drawing at right)

MAIN SCARP - A steep surface on the undisturbed ground around the periphery of the slide, caused by movement of slide material away from the undisturbed ground. The projection of the scarp surface under the disturbed material becomes the surface of rupture.
MINOR SCARP - A steep surface on the disturbed material produced by differential movements within the sliding mass.
HEAD - The upper parts of the slide material along the contact between the disturbed material and the main scarp.
TOP - The highest point of contact between the disturbed material and the main scarp.
FOOT - The line of intersection (sometimes buried) between the lower part of the surface of rupture and the original ground surface.
TOE - The margin of disturbed material most distant from the main scarp.

TIP - The point on the toe most distant from the top of the slide.
FLANK - The side of the landslide.
CROWN - The material that is still in place, adjacent to the highest parts of the scarp.
ORIGINAL GROUND SURFACE - The surface that existed before the movement which is being considered.
LEFT AND RIGHT - Compass directions are preferable in describing a slide, but if right and left are used, they refer to the slide as viewed from the crown.

The following definition of a landslide has been adopted for use in this book:
Landslide - The term "landslide" denotes downward and outward movement of slope-forming materials composed of natural rock, soils, artificial fills, or combinations thereof.
Landslides move along surfaces of separation by falling, sliding, and by flowing. Parts of a landslide may move upward while other parts move downward. The lower limit of the rate of movement of landslide material is restricted in this book by the economic aspect to that actual or potential rate of movement which provokes correction or maintenance.

*The type of material involved is classified according to its state prior to initial movement or, if the type of movement changes, according to its state at the time the change in movement. Thus, the Elm slide (Pl. I-l) began as a rock slide and rock fall in bedrock, but at these a flowing type of movement started the material was "unconsolidated" mass of extremely rapidly moving rock fragments.
**By debris is meant natural soil and rock detritus.

Approximate ranges of rates of movement are according to the scale below

PLATE I
CLASSIFICATION OF LANDSLIDES
Highway Research Board Landslide Committee

REPRODUCED FROM
H.R.B. BUL. 29 1958

T4 S R5E W.M.

SECTION 18

SECTION 19

OREGON FH 55
CLACKAMAS HIGHWAY
LANDSLIDE
VICINITY STA. 425 ±