AN ABSTRACT OF THE THESIS OF

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Title: Mechanics and Movement of the Lookout Creek Earthflow

Abstract approved: ______________________________
Marvin R. Pyles

The Lookout Creek Earthflow is located in the Cascade Mountain Range in western Oregon. The Cascade Mountains are mainly volcanic in origin, and deposits in and around the slide have a complex geomorphic history, affected by glacial, mass movement, and fluvial processes. The currently moving land mass is about 1600 ft. long and 300 to 800 ft. wide. The average depth to the failure surface is around 20 to 25 ft., and the average surface inclination is 9°. Movement takes place mainly in the wet winter months, from late fall to early spring. Average surfical movement is about 4 in. (10 cm) per year, with a maximum recorded velocity of 0.16 in/day (0.4 cm/day). There is some variation in movement rates measured up and down the slope.

Movement is best described by a shear failure of soil at residual shearing strength. Rheological models cannot explain the variations in movement rates with very slight changes in stress levels. Limit equilibrium methods with infinite slope and two-dimensional non-circular procedures are used in backanalysis for strength

parameters. For both methods, back-calculated ϕ' is about 15°, much lower than ϕ' determined from laboratory testing.

Phreatic level observed in three continuously recording piezometers does not correlate well with the annual start of slide movement, or movement rates. The annual start of movement is better explained by the pore pressure-volume change relationship in a 10 in. thick relatively impermeable shear zone.

The Lookout Creek Earthflow is covered by coniferous forest vegetation which is equivalent to a surcharge load of 10 psf or less. There is no effect of root strength along the slip surface of this slide. Removal of vegetation should have very little affect on stability of the Lookout Creek Earthflow.

Mechanics and Movement of
the Lookout Creek Earthflow

by

Keith A. Mills

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MECHANICS AND MOVEMENT OF THE LOOKOUT CREEK EARTHFLOW

I. INTRODUCTION

The mass-movement of soil is a common process in the western Cascade Mountain Range of Oregon. This mass-movement may vary in rate from creep of less than 0.1 in./yr. to extremely rapid slides moving faster than 10 ft/second and in size from less than 1 yd^3 to over 1,000,000 yd^3. Because of these great variations, it is difficult to study the entire mass-movement process. However, a detailed study of one such feature may lead to more knowledge of hillslope movement in general. The purpose of this research was to investigate the mechanics and movement of one mass-movement feature, a slide which is termed the Lookout Creek Earthflow.

The term earthflow itself is probably not sufficient to classify a landslide for engineering purposes. Mass movements which are called earthflows move at rates between 0.02 and 240 meters per year (Swanson and Swantson, 1977, Zaruba and Mench, 1982). Variability in the physiography of "earthflows" is immense. Many landslides which are termed earthflows have a definite basal shear surface, and move like a block slide (Kelsey, 1978). Other earthflows consist of unconsolidated soil and debris, with shear strain distributed fairly evenly through the mass, and have shapes which adapt to the relief of the ground surface.

Previous work on the Lookout Creek Earthflow concentrated on surface morphology and surface deformation. Instrumentation was set up to monitor rates of displacement across shear and tension cracks, water input, and groundwater levels. Initial work showed that average yearly movement was about 4 inches (10 cm) per year, a very slow mass movement (Varnes, 1978). However, this rate is at least 10 times greater than movements in the area associated with downslope creep, and there appeared to be definite boundaries of this earthflow.

This mass movement was classified an earthflow based on general surficial morphology and rate of movement (Swanson and Swantson, 1977). Several reports have been published referring to this mass movement as the Lookout Creek Earthflow, so it has become the accepted name of this land form. Regardless of the findings of this investigation on the mechanics of this mass movement, this slide will be referred to as the Lookout Creek Earthflow throughout this paper.

This report presents the results of an engineering analysis of the Lookout Creek Earthflow. For most engineering projects, the purpose of engineering analysis of slopes is to examine the factors affecting overall stability in terms of a factor of safety. In general, the greater the factor of safety is above 1, the more stable the slope. In the case of currently active or moving slides, the results of engineering analysis are generally used as a guide in efforts to stop further sliding. Little engineering research has been done on the mechanics (including dynamics) of moving slides, because the goals of engineered works are either to prevent or to stop movement, not to control or predict the rate of movement.

The purpose of this study was to make a complete engineering analysis of the Lookout Creek Earthflow, and investigate the relationship between slide mechanics and movement. Included in this report are the results of the following endeavors:

1) Review of previous work on this earthflow and other similar features,
2) Reconnaisances of the site, including geologic investigation, topographic mapping, and determination of climatic and vegetational relationships,
3) Instrumentation of the earthflow to determine movement and groundwater relations,
4) Sampling of slide material relevant to the shearing process,
5) Testing of sampled soils for relevant engineering and physical properties,
6) Stability analysis of the earthflow including backanalysis of existing conditions, and
7) Comparison of the results of backanalysis with engineering properties determined from laboratory tests, and with stability analyses results from adjacent stable ground.

The objectives of this study were to evaluate the results of this standard geotechnical engineering analysis with respect to applicability to earthflow mass-movements in a forested environment. In more specific terms the objectives were as follows:

1) Quantification of movement rates throughout the slope, including variations across, up and down the earthflow, with depth, and with time.
2) Evaluation of the advantages and limitations of various types of field instrumentation in the monitoring of relevant data in an earthflow mass-movement.
3) Determination of the effects of forest operations such as harvesting and road building on the timing and magnitude of earthflow movement.
4) Comparison of laboratory test results on samples obtained from the earthflow with observed conditions and parameters determined from slope backanalysis.

II. SITE DESCRIPTION

The Lookout Creek Earthflow is located about 55 miles east of Eugene, Oregon, in the H. J. Andrews Experimental Forest at approximate Latitude North 44° 15' and Longitude 122° 09' West. Figure 1 shows the location of the Lookout Creek drainage basin. The general physiography of the area is shown in Figure 2. This Lookout Creek basin is located in the western Cascades geologic province, an area of maturely dissected, mainly igneous rock formations.

Slide Morphology

The currently active moving area is approximately 1600 ft. long, and between 300 to 800 ft. wide, as shown in Figure 3. The average ground slope on the surface of this slide is 9°, though there is a considerable amount of topographic variability. The ground surface is very uneven, with as much as 35 ft. change in relief in a horizontal distance of 60 ft. The toe of the slide is bordered by Lookout Creek. The alignment of the creek appears displaced to the south, away from the toe.

A shear crack appears to bound the west side of the Lookout Creek Earthflow. This crack occupies a depression which varies between 2 and 15 feet below the surrounding ground. The shear/tension crack bordering the north portion of the moving mass is much less distinct, and there is little or no surficial cracking evident along the east side. Within the main slide mass there is at least

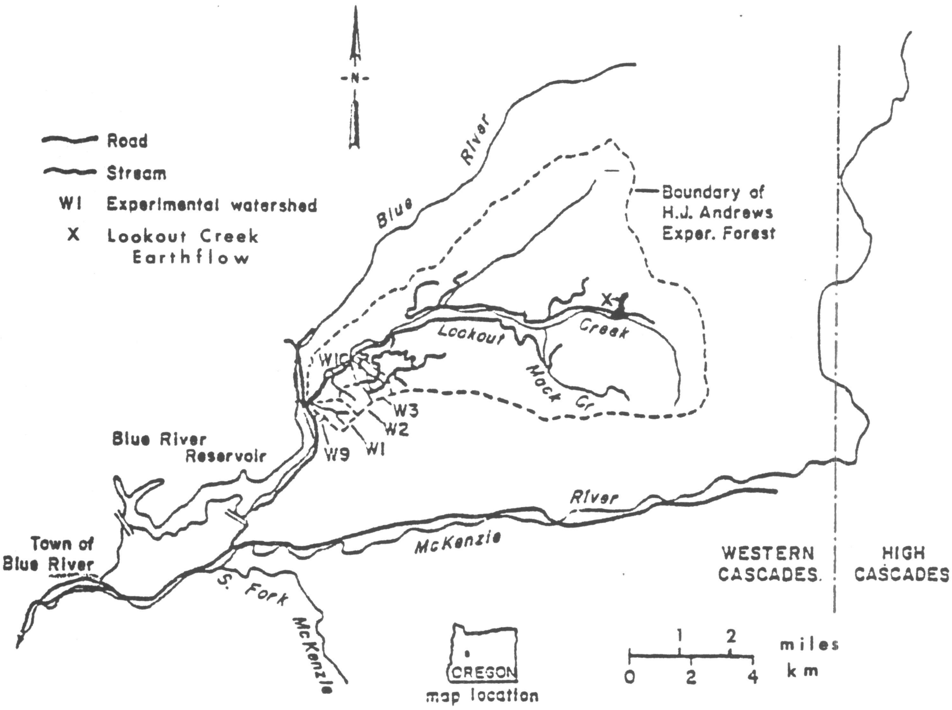

Figure 1. Study area location (after Swanson, Horr, Fredriksen, 1979).

Figure 2. Physiography and vegetation in the Lookout Creek Drainage Basin.

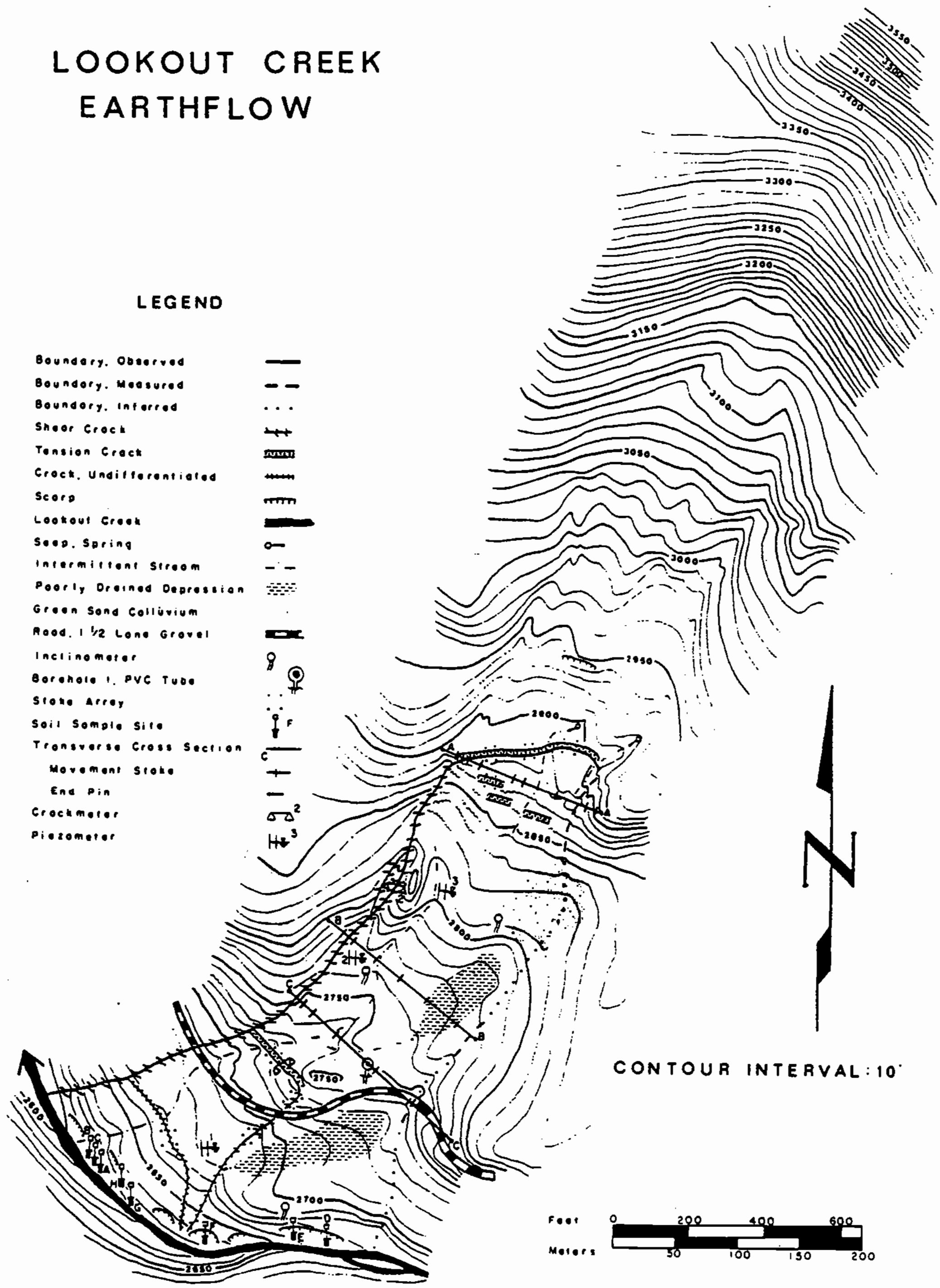

Figure 3. Topography Map

one distinct block which is surrounded by surficial cracking, and there are many other cracks visible near the slide margins except along the eastern edge.

The surficial soil deposits on the Lookout Creek Earthflow consist mainly of red-brown sandy silts with large (3 in. to greater than 2 ft. diameter) subrounded to angular rocks. Most of these rocks are igneous flow rocks, though there are a few boulders (up to 8 ft. diameter) of pyroclastic breccias. Near the northeast corner of the slide there is a deposit of weathered green sands and silts. Most of the trees in this area are dead and partially buried by the green sand and silts, so these soils may have been deposited from a recent small debris flow. Just above this green soil deposit there is a green, highly weathered rock outcrop which lines both sides of a deep gorge. The gorge is 20 ft. deep in spots, and about 250 ft. long, extending from just inside the north slide margin upslope to the north. Near Lookout Creek, at the toe of the slide, there are alluvial deposits, and some exposures of fine grained soils.

The drainage on this mass-movement is best described as discontinuous. There are several sets of parallel intermittent streams. In these streams, water may flow for a hundred feet, then disappear, and then reappear further down in the same, or a parallel channel. There are many seeps and springs, especially near the north boundary. Depressions and swamps are distributed throughout the moving mass, and may be as large as 0.7 acres, and contain water to depths of about 3 feet.

The only major change in the morphology of the Lookout Creek Earthflow due to non-natural processes has been the construction of a 1 1/2 lane gravel road. This road varies in width from 20 to more than 30 ft, and in places there may be as much as 10 ft of fill, or 10 ft cut into existing slopes. The effects associated with any instrumentation and other research activities have been very minor.

Climate

The climate of western Oregon is greatly influenced by North Pacific maritime currents, and characterized by wet, mild winters and cool, dry summers. The typical storms are of low intensity and long duration, with a recorded maximum storm precipitation of 13 in. in 4 days, at an elevation of 2000 ft. in the Andrews Experimental Forest. Actual precipitation at the slide study site has been measured only as water input (where snow is only measured as it melts). There is a good correlation between Watershed 2 precipitation and water input measured on the Lookout Creek Earthflow (see Figure 1). Total precipitation at Watershed 2 is within 2 percent of the measured water input on the slide, so precipitation data from Watershed 2 was used in the investigation. Based on this information, average annual precipitation on the Lookout Creek Earthflow is around 90 in. (230 cm), with 85% falling between October and April.

Elevation has a very significant effect on temperature, and amount and form (rain or snow) of precipitation. The elevation of the active slide varies from 2600 to 2900 ft. This is a zone of transient winter snow cover resulting from a complex winter precipitation regime of snow, rain, and rain on snow events, which is variable over time of year, elevation, and topography (Harr, 1981). Winter snow cover on the study area may vary from none to perhaps as much as 10 ft. with 2 1/2 ft. water equivalency. Mean January temperature in the study area is approximately 32° F (0° C), and mean July temperature is about 65° F (18° C). Daily temperature and cumulative precipitation between 1980 and 1983 are presented in Appendix A.

Vegetation

The climate at the lower elevations of the Western Cascades is very supportive of a dense coniferous forest, which covers most of this region. Major tree species on the Lookout Creek Earthflow include Douglas-fir (*Pseudotsuga menziesii*), western red cedar (*Thuja plicata*), and western hemlock (*Tsuga heterophylla*). Other large conifers growing on the slide are mountain hemlock (*Tsuga mertensiana*), grand fir (*Abies grandis*) and a few noble fir (*Abies procera*). Common understory species on the slide are: vine maple (*Acer circinatum*), which appears to be the dominant understory species, Pacific rhododendron (*Rhododendron macrophyllum*), red huckleberry (*Vacinium parvifolium*), wild blackberry (*Rubus vitifolius*), bracken fern (*Pteridium aquilinum*), and various grasses. Understory species in the poorly drained depressions include devils club (*Oplopanix horridum*), willow (*Salix spp.*), and horsetail (*Equisetum spp*).

The distribution, size, and age of vegetation on and around the slide are influenced by ground movement and related changes in groundwater, external factors including fire history, forest management activities, and many other site specific factors. Forest fires burned over this area in the mid-1800's, but many of the older trees survived and show fire scars. The largest trees which survived the fire(s) are mainly Douglas-fir and western red cedar, now 300 to 500 years old, ranging in diameter between 40 in. to 90 in., and up to 200 ft. tall. Only a few scattered fire surviving trees remain in the area just above the currently active slide, where the most common species is western hemlock, up to 150 years old, and generally less than 20 in. in diameter. An area about 20 acres above the actively moving mass was clearcut logged in 1968, and 25 acres on the east boundary of this feature was clearcut in 1961. About 3.5 acres of the currently moving slide is part of the eastern clearcut, and are covered mainly by Douglas-fir up to 25 ft. tall.

Most of the large standing trees on the earthflow are leaning, but there does not appear to be a pattern to the directions. There are many large trees which have fallen over, as shown in Figure 4. Several trees around tension and shear cracks have been split by differential movement, and roots across these cracks appear to be in tension (Figure 5). The growth of scar tissue around tree cracks indicates tree splitting taking place at least 80 years ago (Swanson and Swantson, 1977).

Figure 4. Organic debris on the Lookout Creek Earthflow.

Figure 5. Roots stretched across crack 100 feet above road.

Certain plant species are characteristically found in poorly drained areas but are uncommon elsewhere. Devilsclub and horsetail are found mainly around the poorly drained depressions and at the toe of the earthflow. In these areas, there are few conifers, another indication of high groundwater levels. When the phreatic level is very close to the surface, most large plant species will not be able to survive because of the lack of aeration in the root zone.

Bedrock Geology

The Cascade Mountains of Oregon are chiefly volcanic in origin and of Cenozoic age. There are two major geologic provinces, the western Cascades and the High Cascades. The western Cascades are older, of late Eocene to late Miocene age, maturely dissected, and consist of deformed and partially altered flows and pyroclastic rock formations. The High Cascades are predominantly undeformed and unaltered andesitic and basaltic flows of Pliocene to Recent age. The geologic formations of the Cascade Mountain Range were identified based on a limited number of paleobotanical and petrographic examinations, and stratigraphic position (Peck and others, 1964). Therefore, exact boundaries between geologic units are not known.

The Lookout Creek Earthflow is underlain by rocks of the Sardine formation of middle to late Miocene age (between 10 and 20 million years old). The Sardine formation varies from less than 3,000 to 10,000 feet in thickness, consisting of a series of lava flows between 10 and 100 ft. thick, with pyroclastic interbeds. The major rock types in the flow units are hypersthene andesite, olivine basalt, basaltic andesite, and dacite. The interbed rock types are tuff breccia, lapilli tuff, and conglomerates (Williamson, 1983).

The exact contacts of the Sardine formation with older and younger formations in the area around the slide are not precisely known. Much of the general area is covered by colluvium and dense forest cover. Mapping of bedrock geology in the Lookout Creek drainage is based on predominant rock types, since units designated by one rock type contain lesser amounts of other rock types. A bedrock geology map of the earthflow area is shown in Figure 6 (Swanson and James, 1975). Most of the flows are believed to be nearly horizontal, although the Cascade Mountain Range is slightly downwarped into broad northeast trending folds (Thayer, 1939). The closest measured strike and dip of a flow contact of the Sardine formation, about 2 miles west of the study site, is approximately N 55° E, 8° SE.

The most recent formation in the Lookout Creek drainage basin, which caps most of the ridges above 4000 ft. elevation, has been informally designated the "Pliocascade volcanics" (Swanson and James, 1975). It consists of andesitic and basaltic lava flows, cinders, interbeds of lake sediments, and tuff breccia of Pliocene to Recent age (less than 5 million years old). None of this formation underlies the earthflow. Erosion products from the Pliocascade volcanics from glacial and mass-movement processes, are probably components of the study slide.

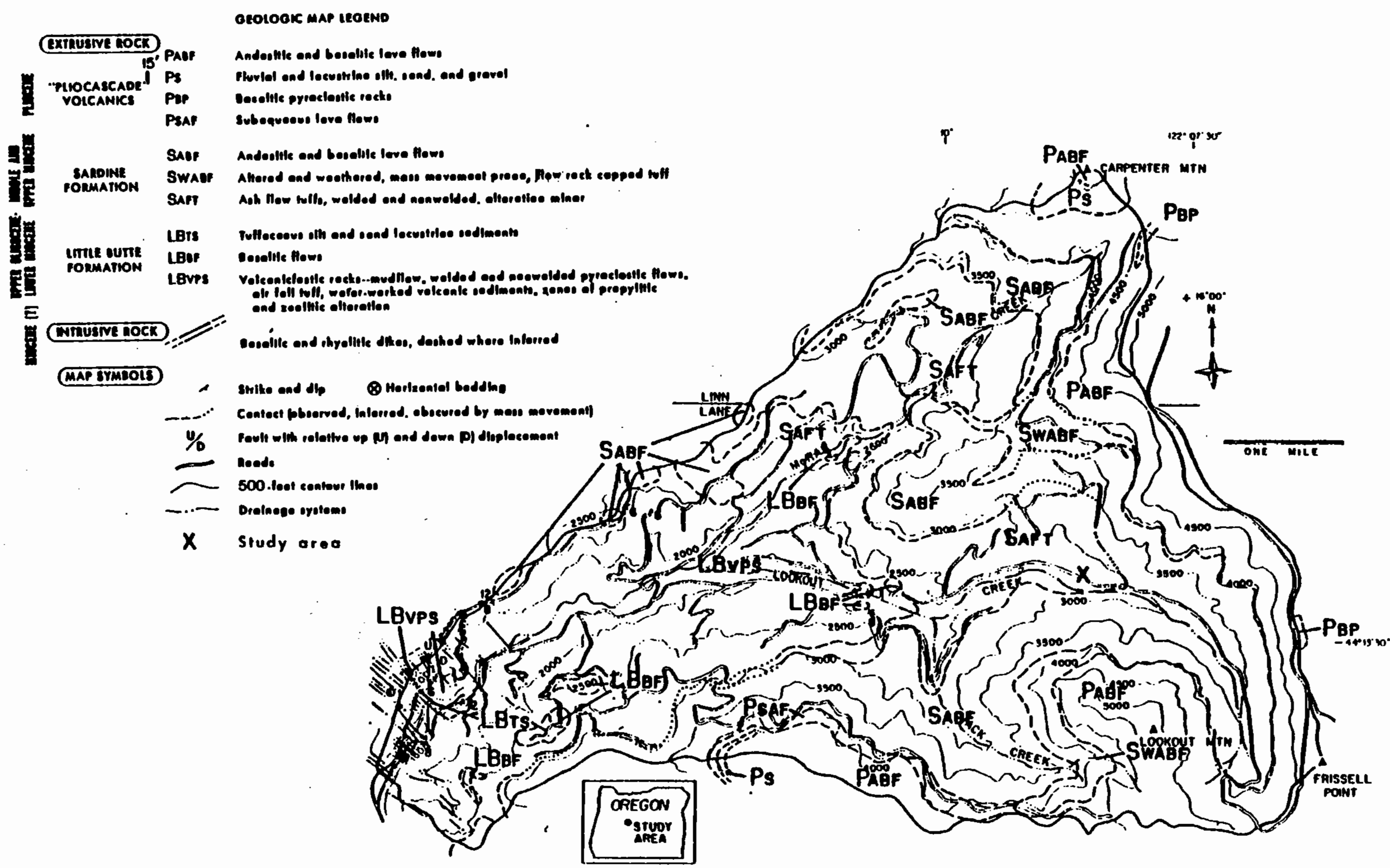

Figure 6. Bedrock Geology Map, H. J. Andrews Experimental Forest (After Swanson and James, 1977).

Geomorphic Processes

The landforms around the study area were formed by glacial, mass-movement, fluvial, and weathering processes. The present landscape has resulted from interactions between all these geomorphological processes. Development of the present landforms began about four million years ago (Swanson and James, 1975). The general landforms and processes were interpreted based on a review of pertinent literature and initial field reconnaissance. Field investigations of specific landforms in the Lookout Creek area have been limited and much of the ground surface is covered by forest vegetation and debris, so the following description of landforms and processes is necessarily brief.

Weathering

Bedrock weathering is very dependent on parent material and hydrologic conditions. The basaltic and andesitic flows are quite resistant and initially tend to be broken along joints by mechanical processes. The resultant boulders and cobbles often form a surface colluvium over much of the ground surface in the study area. Pyroclastic rocks may be less jointed than the Flows, though the rock itself is generally more permeable than lava flows. Chemical weathering is common in pyroclastic materials, especially hydrolosis and hydration.

Water availability is extremely important in the formation of clay minerals. Montmorollinite may be a weathering product of either pyroclastic or flow rock. However, in well drained soils the products required for montmorollinite formation are leached away, and this mineral generally forms where there is little groundwater flow (Birkeland, 1974). Halloysite, a phyllosilicate clay mineral similar to kaolinite, and amorphous clay components (lacking crystal structure) generally are the residual materials from weathering of the surificial flow rock colluvium (Taskey, 1978).

Other common residual weathering products in the western Cascades include gibbsite, zeolite, mica, kaolinite, and chloritic intergrade. Most attempts at soil mapping in this area relate soils to dominant geomorphic process, and actual relationships between soil and weathering of the bedrock may be misinterpreted on these maps (Williamson, 1983). An example is the common term "green breccia" which is interpreted to be pyroclastic rock but may actually be weathered igneous flow rock.

A previous study of clay mineralogy in the western Cascades included soil samples from two areas of the Lookout Creek Earthflow. The main components of these soils are amorphous material, hallosite, and chloritic intergrade (Taskey, 1978). The amorphous material, silt and clay size particles made up of gel, imogolite strands, glass shards and phytoliths, are often called allophane soils. A common structure in these soils consists of gel coatings around clay particles. The physical and mechanical properties of amorphous soils are quite different from other soils, and strength structure relationships are unclear (Maeda, Takenaka, and Warkentin, 1977).

Glacial

Glacial landforms, especially small cirques, are fairly common at high elevations in the Andrews Experimental Forest. Most of the cirques are located on north and northeast aspect slopes where ridge elevations are greater than 4500 ft. (Swanson and James, 1977). Generally, surficial glacial landforms are of Wisconsin age, since older glacial landforms were, for the most part, destroyed by the effects of subsequent glaciation and other erosional processes. Many of the glacial landforms now visible, especially the small cirques, are the result of mountain or alpine glaciation. Valley glaciation was also common in this area, although these landforms are generally much more altered by subsequent geomorphic processes. The valley glaciers also developed at higher elevations, but flowed down along valley bottoms, possibly including Lookout Creek.

A few thousand feet west of the study area, exposed in the scarp of another slide, there appears to be a deposit of glacial till (Figure 7). Glacial ice striations are found as far downstream as Blue River Reservoir, at 1200 ft. elevation and 10 miles from the study area, although these striations were probably formed by glaciers flowing down the McKenzie River basin. A pre-latest Wisconsin glacial deposit forms part of the saddle area on the Blue River-McKenzie River drainage divide (Williamson, 1964). However, most of the geomorphic investigations in this area are aimed at Wisconsin glaciations (between 10,000 to as much as 170,000 years ago) (Gottesfeld and others, 1981, Scott, 1977, Crandell, 1965) though

Figure 7. Glacial deposit 2000 ft.
west of study area.

(Gottesfeld and others, 1981, Scott, 1977, Crandell, 1965) though other Pleistocene glaciations date to 2 million years (Wood, 1977).

Mass-Movement

Mass-movement of soil and rock has had a major impact on landforms of the Lookout Creek area. These deposits may be recognized by characteristic large scale topography such as reduced slope gradients in the slide mass and headwall scarps, and small scale features including disrupted ground surface, unusual surface drainage, and distinctive vegetation. Most mass-movement landforms near the study area, if not stationary, are moving at a creep rate (generally less than 0.4 in per year) (Swanston and Swanson, 1976). Rapidly moving slides are usually obvious, but slow moving earthflows often require careful inspection and measurement to verify sliding motion.

There is some relationship between bedrock geology and large mass-movement landforms. Approximately 25 percent of the land in the pyroclastic designated bedrock units in the Lookout Creek basin is covered by earthflow type deposits, while only about 1 percent of the flow rock formations are covered by these mass-movement terrains (Swanson and James, 1975). However, because of the non-uniform nature of bedrock formations and many other factors, most generalizations about these large mass movements are grossly simplistic. Shallow, rotational, slump-type mass-movements and rapid debris flows are more commonly associated with climatic events and forest management activities than with a specific bedrock unit. In 1978, 48 out of 58 small landslides occurred during rainstorms associated with snowmelt, and many of these were related to forest management activities (Harr, 1981).

Fluvial

The profile of Lookout Creek in the study area is shown in Figure 8. The average stream gradient in this area is a fairly uniform 4 degrees, and stream channel width varies from about 50 ft at the slide toe to 160 ft in some areas above and below the slide (Swanson and Swanston, 1977). Most of the stream bed consists of cobbles and boulders up to several feet in diameter, and in several locations there are large accumulations of logs, branches, and root masses.

The major fluvial effects on landforms occurred during major flood events when there were interactions between fluvial and mass-movement processes. A major flood occurred in this area during 1964, when warm rain on an accumulated snowpack caused a flood event of about 100 year recurrence magnitude. Several small slides larger than 100 yd^3 (75 m^3) were observed where the earthflow toe meets the Lookout Creek channel after this event (Swanson and Swantson, 1977).

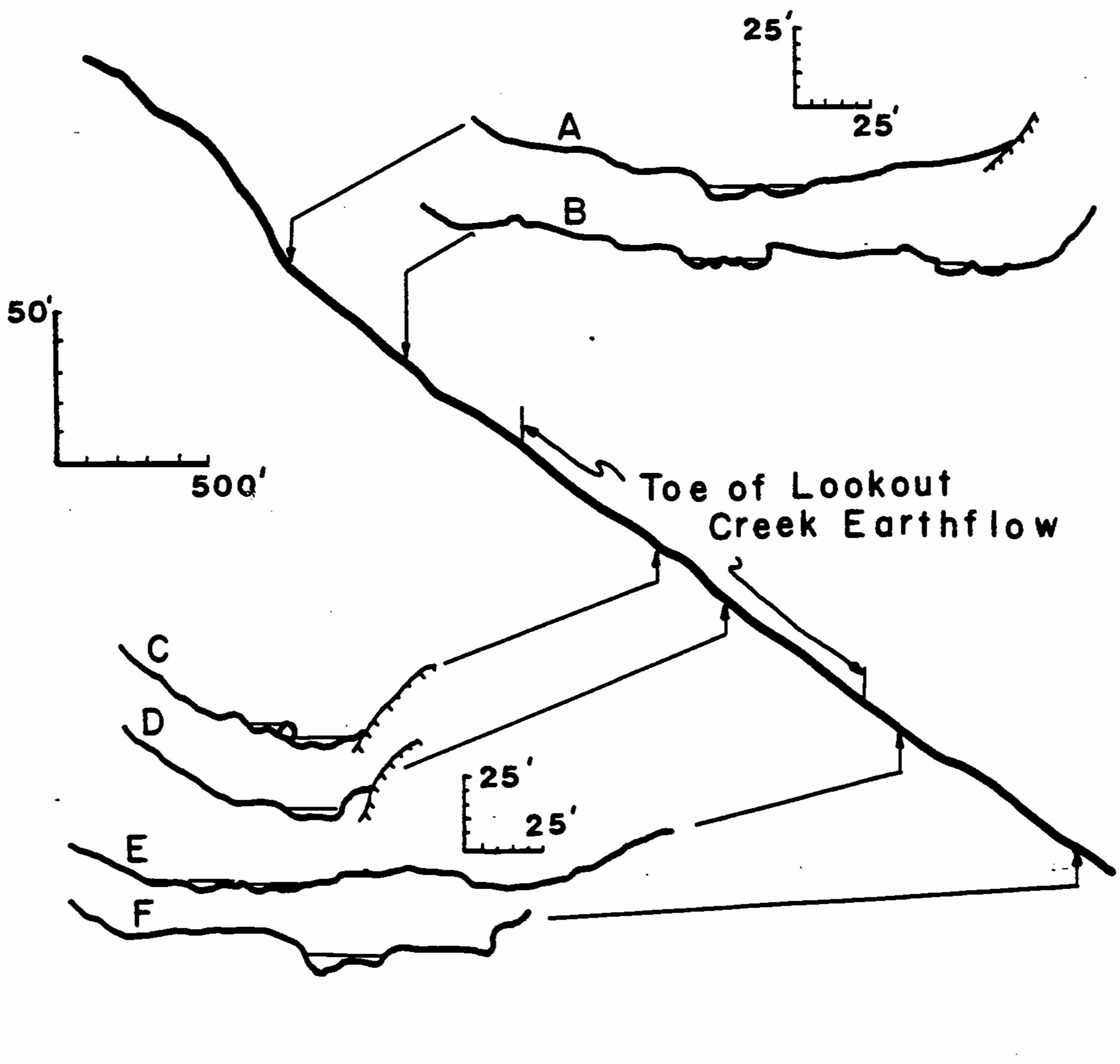

Figure 8. Long profile and cross profiles of the Lookout Creek valley floor in the study area vicinity (after Swanson and Swantson, 1977).

III. FIELD INVESTIGATION AND INSTRUMENTATION

Field investigation and instrumentation are integral parts of an engineering analysis of slope stability. The reliability of any analysis is limited by the scope, completeness, and precision of the field work. A thorough engineering analysis of slope stability requires detailed quantitative descriptions of slope geometry, groundwater conditions, and subsurface materials.

Measurements on the Lookout Creek Earthflow began in the early 1970's, as part of U.S. Forest Service Pacific Northwest Experimentation Studies in the H. J. Andrews Forest. At the present time the Forest Service research includes periodic and continuous measurements of groundwater levels, and surface and subsurface movements. Additional field work required for the engineering analysis described in this paper included a topographic survey, establishment and monitoring of transverse stake sections, borehole drilling, sampling, and limited investigations of groundwater conditions and subsurface movement.

Topographic Survey

The purpose of topographic surveying in a landslide investigation is to quantify the surficial slide geometry and to evaluate surface deformation over time. Most existing topographic maps are at too small of a scale to be used for landslide analysis. For a large landslide the map scale should be no larger than 1:5000, or approximately 1" = 400' (Sowers and Royster, 1978). Surface topography may be determined from conventional surveying with transit and tape by using an electronic distance measuring instrument (EDM) and theodolite, or with aerial photogrammetry and surveying combined.

In heavily forested areas, such as the Lookout Creek area, reliable determination of surface topography from aerial photographic techniques was not possible. Therefore, an EDM and theodolite traverse was surveyed around the study site, and topography was quantified with a stadia survey. The closed traverse around the earthflow used 30 stations, with a horizontal closure of 1:7100 and a vertical closure of 1:800. An open traverse between the U.S. Coast and Geodetic Survey "Lookout" benchmark and a point on the closed traverse was used to establish elevations in the study area. Topography was quantified based on a stadia survey with 450 points, though there was still uncertainty in the final contour map because of the rugged and non-homogenous nature of the ground surface on and around the Lookout Creek Earthflow.

Measurement of Surface Movement

Surface deformations of moving slides can be determined using periodic measurements of specific ground points. Terrestrial photogrammetric techniques can identify movements to an accuracy of 6 to 9 millimeters (Wilson and Mikkelsen, 1978). A transverse cross section, between two points off of the landslide, can show how deformation varies across the slide. Measurement of relative movement in an array of permanent stakes around a crack may be used to determine the local magnitude and direction of crack separation. Other techniques, including surveyed longitudinal profiles and special mechanical instrumentation, may be limited by insufficient precision for reliable determination of surficial movements.

The heavy tree cover prevented the use of photogrammetric techniques for determination of surficial displacements in the study area. To quantify yearly surface movement of the Lookout Creek Earthflow, and how this movement varies spatially, three traverse sections were installed, as shown in Figure 3. For each section, end pins were established on stable ground, on opposite sides of the currently active slide area. Five to eight stakes were set on a straight line between these end pins. The end pins were assumed to be stationary. Surficial creep rates in the areas adjacent to the Lookout Creek Earthflow were not measured in this study. Previous work in the H. J. Andrews Forest found that creep rates are generally less than 0.04 in. per year (Gray, 1977). Therefore, the assumption of stationary end pins should not be greatly in error. Yearly displacements for the water years 1981-82 and 1982-83 are shown in Figure 9.

As part of the U.S. Forest Service study, measurements of stake-array and crackmeter movements are recorded about every three weeks. Up to 12 stake arrays, and 2 crackmeters, are used in the Forest Service studies of the Lookout Creek Earthflow (Swanson, Harr and

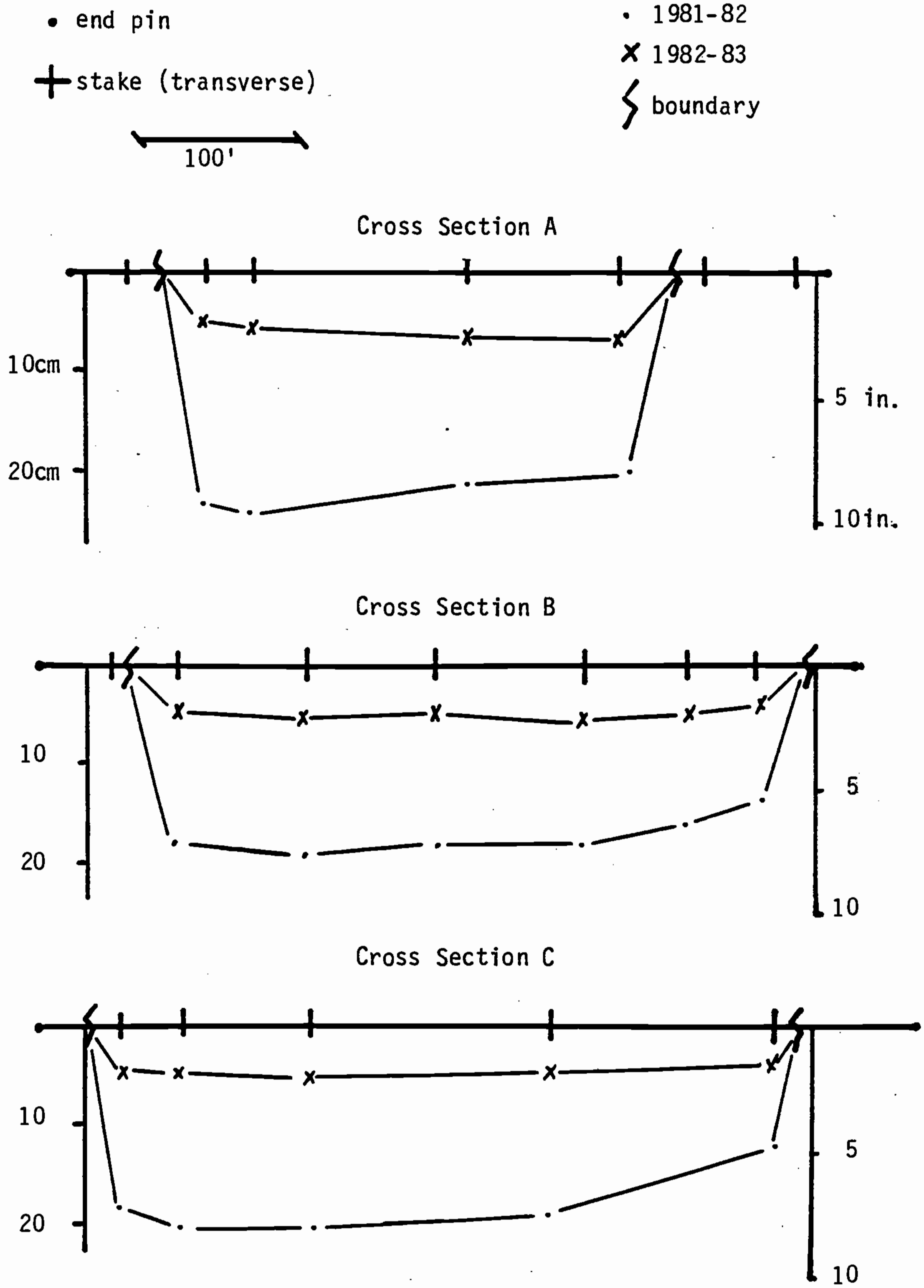

Figure 9. Downslope movement at cross-sections A, B, and C.

Fredriksen, 1979). Yearly surveys of a longitudinal profile are also used in an attempt to detect compressional and extentional movements.

With the exception of crackmeter measurements, a continuous record of surface movements is not possible with any of the measurement techniques previously listed. By using a crackmeter, as shown in Figures 10 and 11, a continuous record of surficial displacement between two points is obtained. This apparatus consists of a tape or wire which is attached on one side of a crack, stretched across the crack, and attached to a recording device on the other side. With the use of a battery powered recording gage, the crack separation over time may be determined. This measured crack separation is usually different than average surficial movement, because of edge effects, alignment of crackmeter, and other factors. Crackmeter 2 (Figure 3) data were correlated with stake array measurements, and are shown in Appendix A. The corrected crack meter data are within 10% of the surficial movement determined from transverse Section B (the middle section).

Subsurface Exploration

Quantification of subsurface conditions and determination of slide material distribution and properties are essential for engineering analysis. If at all possible, it is advantageous to obtain relatively undisturbed samples from the slip surface to be tested for stress-strain and strength properties. Knowledge of groundwater conditions is very important as well, including the location of the

Figure 10. Crackmeter 2 across west boundary tension crack

Figure 11. Instrumentation for recording crack separation.

phreatic surface and quantification of seepage forces. Three methods may be used to determine the subsurface conditions in a slide: geophysical techniques, test pits, and drilled boreholes.

Geophysical Techniques

Geophysical techniques include seismic reflection and refraction, electrical resistivity, and gravitational procedures. It may be possible to locate the phreatic surface, and, in some cases, the failure surface of a landslide, with a geophysical method. However, it is impossible to adequately evaluate soil properties and groundwater flow characteristics with geophysical techniques alone.

A seismic refraction survey was attempted on the Lookout Creek Earthflow, but meaningful interpretation of the data was not possible. This may have been due to the disordered, non-layered nature of the soils. Even in the best situations, with uniform, layered deposits over bedrock, geophysical techniques are only a supplement to further subsurface exploration.

Exploration Pits

Exploration pits are usually excavated by backhoes; and, therefore typically limited to depths of 10 to 15 ft. On most large slides, the failure surface is likely to be below the bottom of a hole of this depth. Even if a test pit is excavated through the failure surface of a slide, identification of this failure surface or zone is still difficult. A piezometer may be installed in a test pit before it is backfilled; however, because of the disturbance caused by excavation, the groundwater information obtained may not be very reliable.

Because of the above limitations, the relatively great cost, and the environmental constraints in a forest research area, small test pits at the toe of the Lookout Creek Earthflow were used to obtain soil samples. A total of six holes were excavated with shovels and picks to depths of 1 to 4 ft. The pits were located between 10 and 75 ft. from Lookout Creek, as shown in Figure 3.

Boreholes

There are two different operations in the drilling of boreholes for geotechnical investigations; advancing the hole, and sampling of the soil or rock. Methods of borehole advance commonly used for geotechnical exploration include hand augering, rotary hollow stem augering, rotary cutting, and rotary coring. The type of drilling to be used depends on depth of bore-hole required, nature of subsurface materials, and desired sampling technique (Lowe and Zaccheo, 1975). In cases where the soil is moderately stiff and contains few boulders, hand augering or use of a small rotary drill may be satisfactory. However, where there are many boulders, or in cases of soils with little cohesion, more involved drilling operations are required.

On this slide, the soils contained a wide range of gravel to boulder size particles (1/4" diameter and larger), and depth to bedrock was unknown. Hand augering was not even attempted, and both augering and rotary cutting with an 8 horsepower trailer mounted drill proved unsuccessful. During this drilling operation, small rock fragments would fall back into the borehole above the cutting head, and other rocks would simply be pushed and not cut by the boring process. Because of these and other problems associated with drilling through the Lookout Creek Earthflow, it was necessary to use a large (Acker Mark IV) rotary drill. The borehole was advanced by continuous coring, also known as wire line drilling.

Wire line drilling is very effective for drilling through non-homogeneous mixtures of soil and rock. The drill rod acts also as casing for the hole, and subsurface materials are brought to the surface in the form of a core inside a 5 foot long core barrel of about 1" inside diameter. The core barrel is mechanically attached to the cutting head during the drilling operation and brought up after drilling has progressed five feet.

The typical sample contains most of the cored rock, however, a great deal of soil is often lost or washed away in the coring process. Relatively undisturbed samples are needed for a geotechnical investigation, but obtaining such samples during wire line drilling is very difficult. The undisturbed sample diameter is limited by the diameter of the core barrel, and soils near the drill head are susceptable to drill related disturbance. Recovery of undisturbed samples with any other method of borehole advance would also be doubtful when soil conditions are similar to those at the study site.

With wire line drilling, at least it is possible to advance a borehole through this material.

One borehole was drilled to a depth of 151 ft. with the Willamette National Forest Acker machine, drilling personnel, and geologic supervision. The drilling operation took two days in December, 1982, and a total of about ten man days with the associated crew. The borehole log for the Lookout Creek slide is given in Appendix B, and the recovered core is shown in Figure 12. Many rock fragments (1 in.-36 in. long) were brought up in the core, but in-place bedrock was not encountered. Unfortunately, much of the soil was lost in the drilling process, and the red tags in Figure 12 indicate missing core, which was often soil. However, enough soil was recovered to classify the sand size and smaller (less than 1/4" diameter) fraction of this soil at some depths, as shown in the log.

A sample of wood was brought up with the core from a depth of 77 ft., or elevation 2668 ft., and was subsequently carbon-14 dated to be greater than 40,000 years old. At a depth of about 120 ft., what was believed to be old stream alluvium was encountered. Upon completion of the borehole, a 1 in. inside-diameter PVC tube was installed to the bottom of the borehole, for subsequent deformation and groundwater measurements.

Sampling

To determine the physical and engineering properties which are important for understanding the movement of a slide, it is necessary

Figure 12. Photo of recovered core
from borehole number 1.

to obtain samples of slide material for subsequent testing. Although there is always some disturbance associated with the sampling process, it is often possible to reduce this disturbance to a minimum. If sample disturbance is small, the properties determined from laboratory testing may approach that of the insitu (in its original place) soil. For determination of engineering properties of strength, stress-strain relationships consolidation, and permeability, physical disturbance of soil structure must be reduced as much as possible. Soil samples to be used for simple physical property tests are often acceptable even with a great deal of structural disturbance, but field composition and moisture must usually be maintained.

Samples from the borehole were in the form of a discontinuous core. This was not satisfactory for strength or stress-strain testing; however, important physical properties can be determined with these samples. Because undisturbed soils were not obtained from the borehole and no large test pits were excavated, samples were collected from small test pits in the toe area of the Lookout Creek Earthflow. Plastic, fine-grained soils were exposed in several areas adjacent to Lookout Creek. Relatively undisturbed samples were obtained by pushing 3 in. diameter Shelby tubes and by trimming various sizes of block samples from these soil deposits, as shown in Figure 13.

Subsurface Movement

To determine the location and nature of the basal shear surface of a moving landslide, it is usually necessary to install a flexible tube down a borehole. By measuring the subsequent distortion of this tube, it is often possible to determine the nature of deformation within the landslide mass. Inclinometers are electronic sensing devices used to find the displacement at depth by measuring the inclination of a grooved tube. It is sometimes possible to locate a slip surface with only a weighted rod, lowered down a flexible tube in a borehole till a break or bend prevents passage of the rod. On slides with a single, well defined failure surface, the use of a weight attached to a string may be superior, as there are no complicated electrical or mechanical parts to break down. However, slides with several slip surfaces, or with indistinct zones of deformation, probably require an inclinometer to quantify slide movement.

There are three inclinometer tubes installed in the study area as part of the U.S.F.S. earthflow research. (Figure 3). The depth from ground surface to the bottom of the tube varies from 17 to 25 ft. Deformations calculated from inclination measurements in the three tubes show only very small movements, on the order of a few hundredths of an inch per year. This small displacement is distributed fairly evenly with depth in the inclinometer tubes. This indicates that the inclinometer tubes do not penetrate the slip surface.

Figure 13. Excavation of soil samples where the slide borders Lookout Creek..

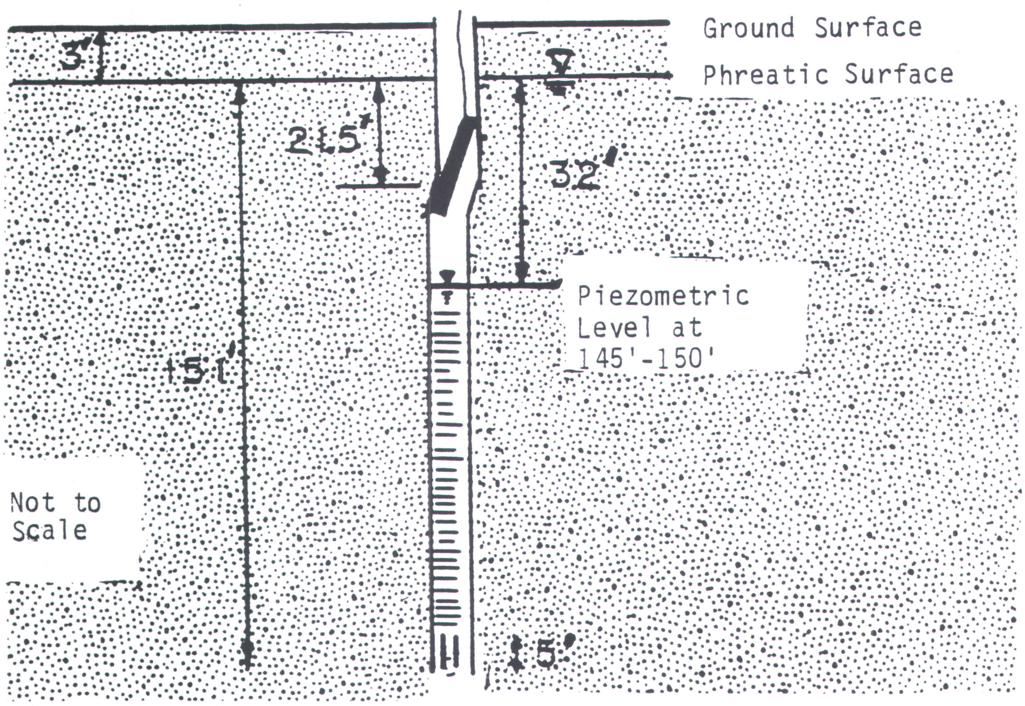

Figure 14. 1 in. PVC tube in Borehole No. 1.

Deformation in the 1 in. PVC tube was used in an attempt to determine the depth to the slip surface and thickness of the shear zone. Measurement of deformation was made in May 1983, 6 months after tube installation. Over this period, downslope movement determined from transverse cross section data was 2.15 in. A 1/2 in. diameter steel rod, attached to a long cord, was lowered down the tube until it would drop no further. Three different lengths of steel rod were used, 4 in., 10 in., and 30 in. The 4 in. rod went to the bottom of the borehole, 151 ft., the 10 in. rod stopped at 21.5 ft., and the 30 in. rod stopped at a depth of 21.2 ft. below the ground surface. These data combined with measurement of surface movement at the borehole, are indicative of slide thickness and deformation characteristics. Tube measurements are shown in Figure 14.

Groundwater Monitoring

Piezometers are instruments used for determination of subsurface pressure and sometimes seepage conditions. Standpipe piezometers usually consist of a vertical tube extending to some depth below ground, sealed above and sometimes below a porous tip on the end of the tube. Piezometers are installed near the inclinometer tubes at three locations in the study area, all as shown in Figure 3.

The piezometers used on the Lookout Creek Earthflow consisted of PVC tubes with lower ends perforated and covered with nylon screen. The hole around the PVC tubes was backfilled with sand, and a bentonite seal was placed above the perforations. Detail of a typical piezometer is shown in Figure 15. Initially, three continuously recording piezometers were used to monitor groundwater changes. Groundwater conditions at Piezometer Number 2, near the center transverse stake section, and Piezometer 1, 150 ft. south of the road, were virtually identical. Piezometer 3, near the upper sag pond, did not show as much variation with water input as the other two piezometers.

The water level recorders use a gas system driving a mercury manometer attached to the drive pulley of a Belfort FW-1 water level recorder (Figure 16). The recorders detect changes in water level as small as 0.2 in. over time periods as short as 15 minutes. Piezometric levels for Piezometer Number 1 are given Appendix A. Note that the difference between minimum and maximum piezometric level is only about 3 ft.

The PVC tube installed in the borehole also acted as a piezometer and registered the pore pressure, or piezometric head, at a depth of 145 ft. The initial piezometric level measured after tube installation was 21 ft. below ground surface. Subsequent measurements have indicated that the actual piezometric surface was generally around 32 ft. below the surface. The first measurement was likely recorded before the water level in the tube reached equilibrium with the groundwater conditions at depth.

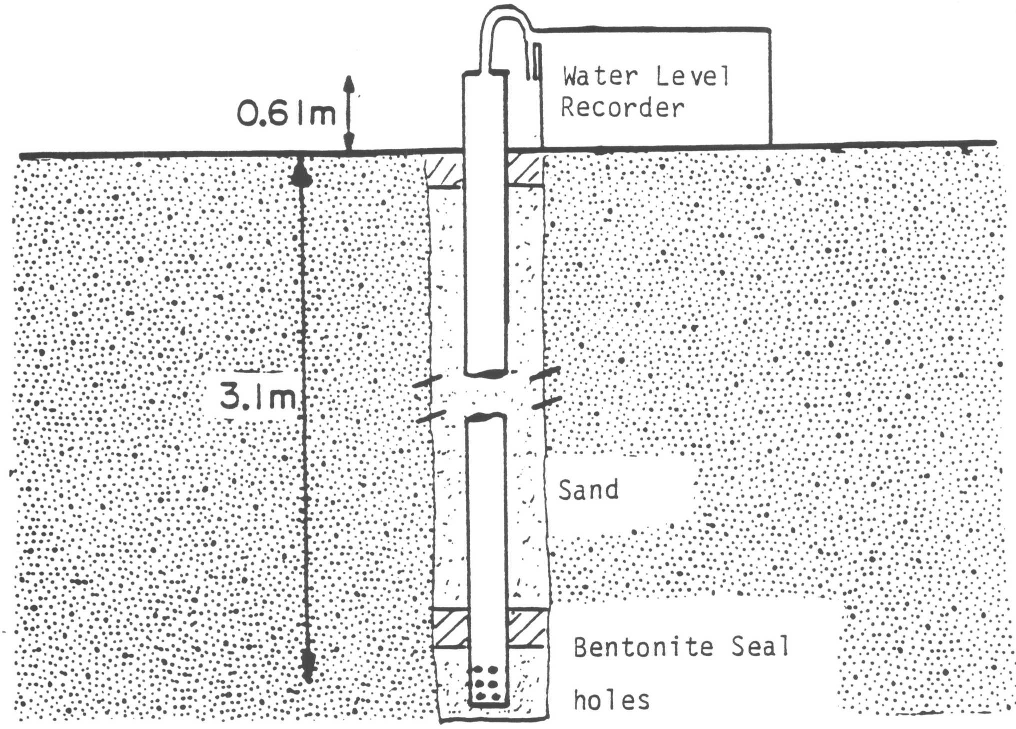

Figure 15. Detail of Piezometer No. 1.

Figure 16. Instrumentation for recording groundwater levels.

IV. LABORATORY TESTING

The determination of soil properties needed for engineering slope stability analysis usually requires laboratory testing. The fundamental engineering properties of soil (permeability, compressibility, shear strength, and stress-strain relationships) are very important parameters for slope stability analysis. Physical properties, including mineral composition, structure, and density, among others, are more important for identifying slide materials. Index properties of the soils are useful for predicting certain engineering and physical properties. Soil samples from the toe of the Lookout Creek Earthflow, bore hole no. 1, and selected surface locations were examined in the laboratory.

Physical Properties

The physical soil properties which influence fundamental engineering properties in fine grained soils are void ratio, structure, nature of bound water, and clay mineral composition. Except for void ratio, these properties are fairly difficult to measure, and there are no exact relationships between engineering properties of a typical soil and its physical properties. The importance of physical soil properties in a landslide investigation is that they are useful for correlation. There is so much variation in mass-movement characteristics that physical soil properties may be very useful in helping to describe a particular mass movement.

Identification of clay minerals is very useful for general categorization of fine grained soils. Since the Lookout Creek Earthflow is located in a volcanic formation, identification of amorphous constituents derived from volcanic ash is important. The unusual properties of amorphous materials could influence stability of the Lookout Creek Earthflow. For determination of clay minerals with crystal structure, the X-ray diffraction technique was used. Oxalate-extraction and water retention on drying tests were conducted to measure allophane content in soils from the toe area and Borehole No. 1. Other important physical properties, which are used with the results from engineering property tests, are the specific gravity of soil solids and the field water content of the soil.

Oxalate Extraction

The oxalate-extraction method was used on soils from the toe of this mass-movement and a sample from Borehole 1, to determine amorphous clay content. An acid oxalate solution was used to dissolve allophane, some immogolite, iron oxides, and organic compounds. Most of the aluminum, silicon, and iron in the amorphous materials is dissolved by the acid oxalate. The fraction in soil of these dissolved components can then be determined (Parfitt and Hemni, 1982). Allophane soil properties, including water holding capacity, change irreversably on drying, so percent water was measured in both air dried and moist soils to determine the effect of air drying. The results of these tests did not indicate high amorphous content.

X-Ray Diffraction

The X-ray diffraction technique is commonly used to identify crystalline clay minerals. Most clay minerals have a distinctive basal spacing (distance between the basic structural sublayers). This basal spacing can be determined by measurement of the diffraction of X-rays of known wave length (Spangler and Handy, 1982).

X-ray diffractions performed on five samples from the toe of Lookout Creek Earthflow in this investigation, and three samples in a previous investigation (Taskey, 1977), indicated halloysite, montmorollinite, some chloritic intergrade, and perhaps some kaolinite in the soils tested (Appendix C). The interpretation of X-ray diffraction data were fairly qualitative, so the amount of each mineral in a soil was not accurately determined.

Index Properties

Index properties of a soil are those which indirectly predict certain engineering properties. For fine grained soils (silts and clays), useful index properties include the Atterberg limits, percent of soil grains smaller than two microns, activity index, and liquidity index, among others. Compared to engineering properties, these index properties are easy to measure, and some very general correlations have been developed between index properties and engineering properties. Knowledge of index properties also helps in planning and preparation for engineering property tests.

Engineering Soil Classification

Engineering classification attempts to relate results of simple tests with soil performance in practical applications. Most classification methods require only grain size analysis and Atterberg limits to classify a soil. To determine the distribution of grain sizes in a soil, a set of sieves is used for coarse grained soils, and a hydrometer is used for fine grained soils. Soils tested from the study area are classified as silts of high plasticity (MH), silts of low plasticity (ML), and sands with fines of low plasticity (SM), based on the Unified Soil Classification System. Atterberg limits are plotted in Figure 17.

Other Index Properties

The amount of soil finer than 0.00008 in. diameter (clay sized)in soils from the toe and Borehole 1 varied from 12 to 38 percent. Liquid limits varied from 47 to 102 percent, plastic limits were between 35 and 75, and plasticity index values varied between 7 and 29. Liquidity index ranged from less than 0 to 0.65, indicating non-sensitive soils. Activity index ranged between 0.22 and 1.20, indicating normally active or non-active soils. The index properties determined from testing Lookout Creek Earthflow soils, and natural soil water content and specific gravity of solids, are presented in Table I.

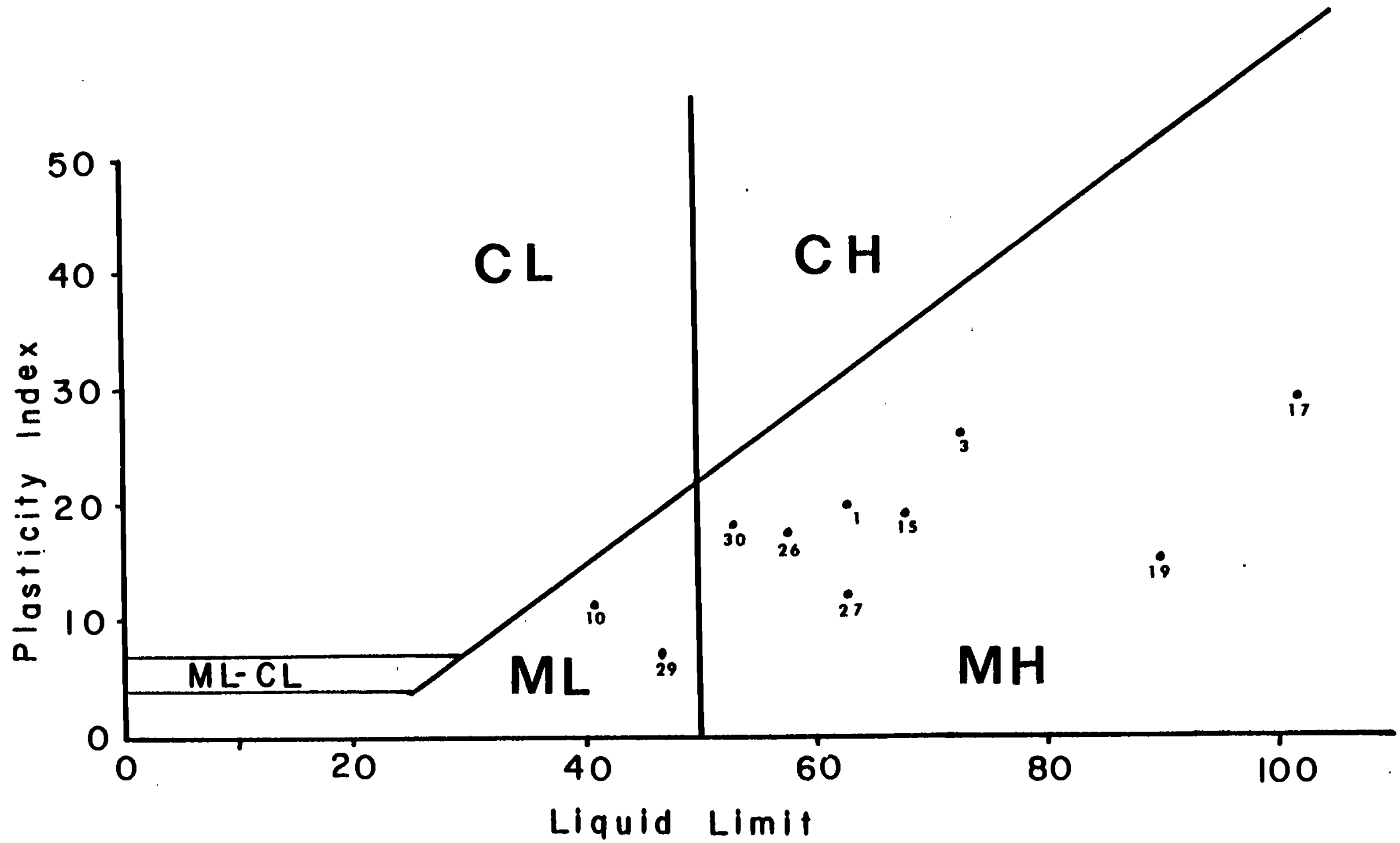

Figure 17. Atterberg limits of soils from the study area.

Table I.

Index Properties of Soils from the Lookout Creek Earthflow

Sample No.	Location	Natural Water Content (W_{nat}) %	Specific Gravity of Solids (Gs)	Liquid Limit	Plastic Limit	Plasticity Index	Unified Classification	Percent -.002 mm	Liquidity Index	Activity Index
1	A	41	2.65	63	40	23	MH	23	-.05	.85
3	D	55	2.70	73	47	26	MH	25	.30	1.04
10	B	37	2.65	41	31	10	ML	10	.40	1.00
15	E	46	2.70	68	49	19	MH	25	-.16	.75
17	C	84	2.30	102	73	29	MH	38	.36	.75
19	Margin	78	2.75	90	75	15	MH	12	.20	1.25
26		44	-	58	41	17	MH	26	.18	.65
27		62	-	63	51	12	MH	22	-.08	.55
29		44	-	47	40	7	ML	31	.57	.22
30		47	-	53	35	18	MH	27	.65	.67

*Borehole No. 1

Engineering Soil Properties

Quantification of relevant engineering soil properties necessitates testing which models the physical processes involved in the field. For this study, soil shear strength parameters were determined from triaxial and direct shear tests. Consolidation and volume change parameters were obtained from consolidation and triaxial testing.

Triaxial Testing

The most versatile laboratory test for determination of engineering soil properties, especially shear-strength and stress-strain relationships, is the triaxial compression test (Bishop and Bjerrum, 1960, Bowles, 1978). The triaxial apparatus used in testing soils from the Lookout Creek Earthflow is shown in Figure 18. The equipment is set up for consolidated-undrained testing. Both volume change during the consolidation phase of the test and pore water pressures during loading are recorded.

All samples tested were 2.8 in. in diameter, and between 5.8 and 6.5 in. long. Sample preparation consisted of extruding the soil from a 3 in. diameter Shelby tube and trimming to the proper length, or carefully trimming the block samples to both the desired diameter and length. The sample was then surrounded by filter paper strips, to permit more uniform and rapid consolidation. After the sample was enclosed with an impermeable membrane, the soil cylinder was loaded into the triaxial apparatus.

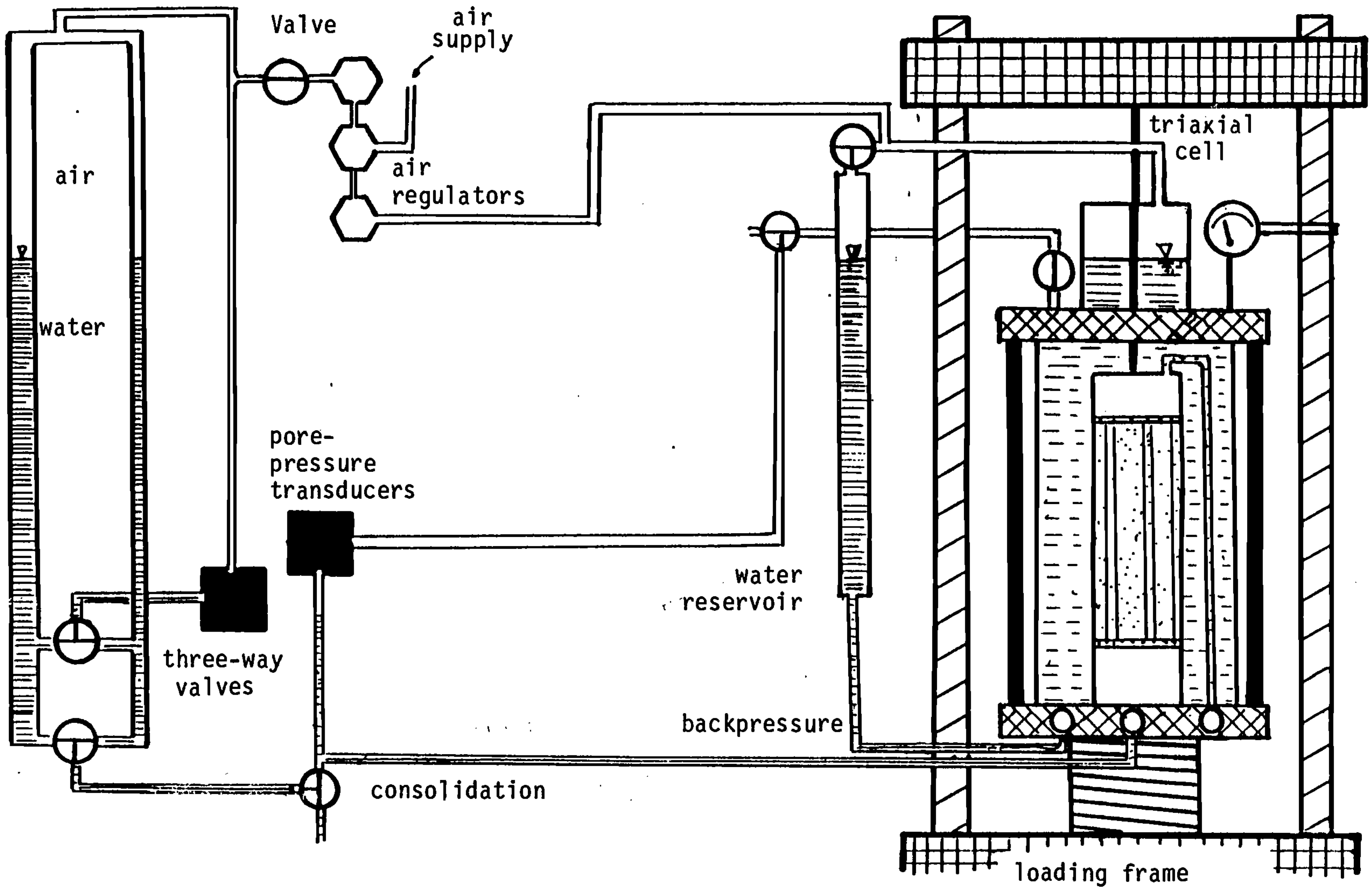

Figure 18. Triaxial cell and volume change apparatus.

Water was used as the confining fluid in the triaxial cell. Determination of the desired strength parameters requires that the tested soil be saturated (soil pores containing only water and no undissolved air). To insure saturation of the soil, a backpressure (water at elevated pressure to force air into solution) and an very slightly greater confining pressure were applied to the sample. Saturation of the sample was evaluated with the B parameter:

$$B = \frac{\Delta u}{\Delta \sigma_3}$$

where u is the incremental change in pore pressure associated with $\Delta \sigma_3$, and $\Delta \sigma_3$ is the increased confining pressure.

The confining pressure, $\Delta \sigma_3$, was increased in 5 pounds per square inch increments until B equals 0.95. When B approaches 1.00, the sample becomes saturated, but because of the effects of soil grain contact area B for saturated soils must be slightly less than 1.00.

The samples were consolidated at confining pressures between 5 and 20 psi. These values were believed to be representative of the confining pressure at the slip surface. Each triaxial sample was consolidated and then loaded three times, a procedure known as stage testing. Stage testing was used because of the limited number of samples available, and the assumed variability between the samples (Filz, 1982).

Loading was strain-controlled, meaning that the loading cap was advanced at a constant rate. The rate of loading was 0.002 in/min or an approximate engineering strain of 0.033 percent per minute. Loading for the first two stages was stopped when the slope of the deviator stress ($\sigma_1' - \sigma_3'$) versus strain curve approached horizontal. Loading beyond this point might have caused a reduction in strength for subsequent sample loadings. With the use of this procedure a reasonable strength envelope should be obtained.

Drainage was not permitted during sample loading, and the pore pressures generated by shearing were measured. This allowed for the calculation of effective stresses, and determination of the effective strength parameters, c', the effective cohesion, and ϕ', the effective angle of internal friction needed for slope stability analysis. The change in pore pressure, Δu, for a given change in stress in undrained loading is given by

$$\Delta u = B\ [\Delta\sigma_3 + A\ (\Delta\sigma_1 - \Delta\sigma_3)]$$

where

$\Delta\sigma_1$ is the change in major principal stress
$\Delta\sigma_3$ is the change in minor principal stress, and A
B are pore pressure parameters.

The B pore pressure parameter was determined to be very close to 1 during backpressure of the sample.

When B equals 1, and σ_3 is constant, the A parameter is given by

$$A = \frac{\Delta u}{\Delta \sigma_1}$$

where Δu is the change in pore pressure associated with a change in total principal stress, $\Delta\sigma_1$. The parameter A is dependent on soil stress history and testing stress level and strain. In general, A decreases with increasing soil overconsolidation and increasing ambient stress. Values for A (at failure) determined from testing of the soils from the toe area of the Lookout Creek Earthflow ranged from -0.05 to 0.41. These values are indicative of a moderately overconsolidated soil (Spangler and Handy, 1982).

Direct Shear Testing

The direct shear test was used in an attempt to determine the residual shear strength values of selected samples from the toe of the Lookout Creek Earthflow. The direct shear test forces a shear failure along a predetermined surface of a soil sample (Bowles, 1978). To model large shear strains on a slip surface, the sample was sheared back and forth up to ten times. The rate of shearing was slow enough to permit at least 90% consolidation drainage, as calculated from triaxial test consolidation values and anticipated loading stresses.

The samples used for direct shear testing were 2.5" in diameter, and between 0.80 and 0.90 inches thick. Loading was alternatively strain controlled, and then stress controlled. The total shear displacement during each loading varied from 0.40 to 0.75 inches. Each sample was sheared under three different normal stresses. Shearing back and forth under each normal stress was continued until there was no further drop in shearing resistance. This was usually observed after one or two loading cycles.

Comparison of Strength Parameters

Triaxial test results from the soils tested indicated ϕ' ranged from 23.2° to 31.0°, and c' ranged between 1.8 to 5.0 psi for eight tests. The range in strength values from the direct shear tests showed $\phi r'$ varied from 27.8° to 35°, and c_r' between 0 to 2.1 psi. These results, especially from the direct shear tests, were somewhat inconclusive. The values of residual effective angle of internal friction, ϕ_r' and residual cohesion, c'_r, should both be less than or equal to the peak values calculated from triaxial tests. This was the case for cohesion, but the friction angle determined from the direct shear tests was even greater than that from triaxial tests. Two possible reasons for these high direct shear measured friction angles were that with direct shear testing a failure plane is forced in the specimen, and testing machine friction may have been significant. Results from the shear strengths tests are given in Table II.

Consolidation Testing

The volume change which occurs after the total stress acting on a soil of low permeability is increased is known as consolidation. Although it is possible to measure consolidation parameters with triaxial testing apparatus, the one-dimensional consolidation test is most commonly used. One-dimensional consolidation theory is much more simple than two- or three-dimensional theories. Consolidation tests on samples from the toe were trimmed to 2.5 inches in diameter and 0.8 inches in height and tested in a floating ring consolidometer. Four consolidation tests were performed on soils from the toe of the Lookout Creek Earthflow (Table III).

To determine volume change relationships during the triaxial testing, an approximate relationship based on isotropic compression was utilized (Akroyd, 1957). For the case of a triaxial specimen with length at least twice the diameter, and set up for radial drainage, the coefficient of consolidation, Cv, is approximated by

$$C_v = \frac{0.524\ R^2}{t}$$

where:

R is the radius of the triaxial specimen,

and t is the time for 90% consolidation.

The results of consolidation tests were plotted as the standard e versus log σ' curve. Using an e versus log σ' curve, it is sometimes possible to determine the maximum previous overburden pressure which has acted on a soil (Terzaghi and Peck, 1967). However, it was not possible to determine this preconsolidation pressure. This may have been due to sample disturbance in the lab or field or the fact that some highly compressible silts exhibit consolidation behavior that is very difficult to interpret for stress history.

Table II

Results from Strength Tests

Sample No.	Site	Type Test	ϕ'^{*}	c' (psi)*
1	A	triaxial	26.0°	2.5
2	A	triaxial	23.9°	2.3
1	A	direct shear	30.0°	1.1
3	B	triaxial	31.0°	1.8
4	B	triaxial	23.6°	2.8
4	B	direct shear	32.6°	1.4
5	C	triaxial	31.1°	5.0
6	C	triaxial	28.8°	3.8
5	C	direct shear	35.0°	2.1
7	E	triaxial	26.5°	2.3
8	H	triaxial	23.2°	2.4
8	H	direct shear	27.8°	0

*direct shear values after shearing back and forth

Table III

Results from Consolidation Tests

Sample No.	Site	Type Test	C_v (in.^{2}/min.)	k(ft./sec)
1	A	triaxial	8×10^{-3}	3×10^{-10}
9	A	consolidation	5×10^{-3}	3×10^{-10}
4	B	triaxial	1×10^{-3}	2×10^{-10}
10	C	consolidation	6×10^{-3}	5×10^{-10}
7	E	triaxial	2×10^{-3}	1×10^{-10}
11	H	triaxial	2.5×10^{-3}	2×10^{-10}

Consolidation and permeability parameters determined from triaxial and consolidation tests included c_v, the coefficient of consolidation, and k, the coefficient of permeability. For soil samples from the toe of the Lookout Creek Earthflow, c_v ranged between 1×10^3 and 8×10^3 in^2/min, and k varied between 5×10^{-10} and 3×10^{-11} ft./sec. This permeability was comparable to that of a highly plastic clay.

V. MECHANICS AND ANALYSIS

The work presented here will consider two possible mechanisms which cause hillslope movement, a shear failure through some three-dimensional surface within the slope, and creep of the slope. In this report, slope stability is evaluated with respect to groundwater levels, minor changes in slope geometry, and surcharge loading. Effects of forest management are evaluated with respect to how they change these factors. Changes in material properties through weathering, erosion, and gradual changes in slope gradient are very difficult, if not impossible, to evaluate for an engineering analysis, and were not considered for this investigation.

Shear Failure

The shearing resistance of soil is a function of a great many factors $S = f\,(e, \phi', C, \sigma', c', H, T, \varepsilon, \dot{\varepsilon}, s)$ where e is the void ratio, ϕ' is the effective angle of internal friction, C is the soil composition, σ' is the effective normal stress, c' is the effective cohesion, H is the stress history, T is temperature, ε is the strain, $\dot{\varepsilon}$ is the strain rate, and s is the structure (Mitchell, 1976). Even though all of these factors are not independent, it is easy to see that an exact quantitative description of soil strength in terms of measurable parameters is not practical.

Shear failure of a soil mass can be defined as the beginning of inelastic action, or the actual rupture of the material (Newmark, 1960). In the case of dense sands and overconsolidated clays failure is generally obvious. However, in loose sands and soft clays, there is no clearly defined failure, and for these soils "failure" must be defined in terms of some design criteria. The Mohr-Coulomb theory is normally used to model the strength of cohesive soils, where S, the shearing resistance, is described by $S = c + \bar{\sigma} \tan \phi$

where: c is the cohesion,

$\bar{\sigma}$ is the normal stress

and ϕ is the angle of internal friction

This equation must be modified for the effects of pore water pressure, as described later in this paper.

Residual Shear Strength

When a soil is strained beyond its peak shear strength, the shearing resistance decreases, ultimately to what is termed the residual shear strength. For some soils, a displacement of many centimeters is required before the shear resistance drops to the residual condition (Mitchell, 1976). Tree scar tissue growth caused by ground deformation indicates that the Lookout Creek Earthflow has been moving for at least 80 years. If the present rate of movement has been going on for the last 80 years, there has been at least 25 ft. (8 m) of downslope displacement. Given this displacement, the shearing resistance of the soils along the slip surface should be at, or very close to, the residual value. The relationship for the residual shear strength of a soil, in terms of the Mohr-Coulomb equation, can be expressed as $S_r' = \sigma_n' \tan \phi_r'$.

where

σ_n' is the effective principal stress, and

ϕ_r' is the residual angle of internal friction.

The residual cohesion is generally very close to zero and can be neglected (Atkinson, 1981, Lefebvre, 1981).

Creep of Slopes

The term creep is used to describe plastic deformation of a material under a constant applied stress. Ter-Stepanian (1963) distinguished three phases during shear loading of soils, rigidity, creep, and failure. The stress required to initiate creep may be less than 30 percent of the peak soil strength. Quantitative relationships for creep require rheological parameters, including the coefficient of viscosity, η, which may not be constant with time, stress, and strain. Rheological models based on stress level and water content have been developed for soil creep and flow, but have very limited field application (Komamura and Huang, 1974).

Creep of slopes, as opposed to sliding, is a more or less continuous process with no well defined boundaries between stationary and moving material (Terzaghi, 1950). This creep of hillslopes may be separated into surficial (seasonal) creep, and depth or continuous creep (Ter-Stepanian, 1963). Surficial or seasonal creep takes place in the zone of seasonal changes in moisture and temperature, with strain being fairly continuous with depth. Based on inclinometer measurements there is little seasonal creep taking place on the Lookout Creek Earthflow.

Depth creep is a more or less continuous process of downslope movement imparted by gravity forces, and deformation patterns more resemble that of a viscous fluid than an elasto-plastic solid (Yen, 1969). Rates of movement in land masses said to be undergoing depth creep range from less than 0.4 in. to over 8.0 in./yr. rates similar to those of the Lookout Creek Earthflow (Ter-Stepunian, 1963). However, models developed to quantify the depth creep process are based on yearly displacements and average stress conditions. Movement of the Lookout Creek Earthflow varies considerably throughout the year, and from year to year. General rheological models do not describe these variations.

Groundwater

Based on the piezometric data from the study area, the soil 10 ft. or more below the ground surface of the Lookout Creek Earthflow is for all practical purposes water saturated. The actual effect of water on slope movement is quite complicated; changes in water levels result in changes in the stresses within the slide mass. These effects are often misunderstood. It is sometimes believed that the lubricating action of water is the major factor causing landslides; in fact, the term earthflow is referred to as "a slow flow of earth lubricated with water..." in the Dictionary of Geologic Terms (American Geological Institute, 1976). This is not at all the case.

Water, in contact with many minerals, often increases the coefficient of friction (Terzaghi, 1950). An extremely thin layer of water is all that is required for full lubrication effects. Therefore, lubrication is not a factor in the movement of the Lookout Creek Earthflow.

The major influence of water on slope stability is the effect of water pressure in reducing the load carried by the soil grains. When a significant amount of water infiltrates into the Lookout Creek Earthflow, whether from a rainstorm, snowmelt, or a combination, the piezometric level rises. This has several effects on the stresses acting within this slide. There is an increase in total unit weight of the soils above the previous ground water level, the additional pore water pressure reduces the effective stress, and there may be a change in the direction of subsurface flow, or seepage angle.

The measured difference between maximum and minimum phreatic levels in the study area is about 3 ft. Average depth of this phreatic surface below the ground surface is between 3 and 6 ft. The degree of saturation, S, of the unsaturated soils, as determined from lab tests, is between 85 and 95 percent. Assuming the average degree of saturation increases 10 percent in the upper 5 ft. of soil during periods of large water input, the total stress at depths below the phreatic surface will increase by less than 15 psf. For this same large water input, the phreatic level increases about 3 ft, so the pore pressures at depth increase by around 185 psf, assuming a seepage angle of 9. $i = \alpha$, of 9° (Figure 19). Therefore, the major effect from an increase in groundwater level is an increase in pore water pressure.

Only part of the total stress acting on a surface below the water table is carried by the soil grains or particles. The rest of the load acts as the pore water pressure, acting equally in all directions. Water, since it is a fluid, deforms continuously when subjected to shear load, and thus the load carried by the water can not contribute to shear strength of the soil-water system. The coefficient of friction of a soil is mobilized only by the stress carried by the soil grains, known as the effective stress, σ':

$$\sigma' = \sigma - u$$

where σ is the total stress at some depth, and u is the pore water pressure at that depth. This equation applies only to water saturated soils, and neglects effects of grain to grain contacts, which have been shown to be very small (Skempton 1961).

For any situation where groundwater is present in a soil, the Mohr-Coulomb equation for soil strength must be modified. This concept was developed by Karl Terzaghi in the 1920's, and the actual soil strength is given by:

$$S = \sigma' \tan \phi' + c'$$

where σ' is the effective stress, ϕ' is the effective coefficient of friction, and c' is the effective cohesion (Terzaghi and Peck, 1967).

When water is flowing through a soil, the frictional drag on soil grains causes an energy loss in the flowing groundwater. This

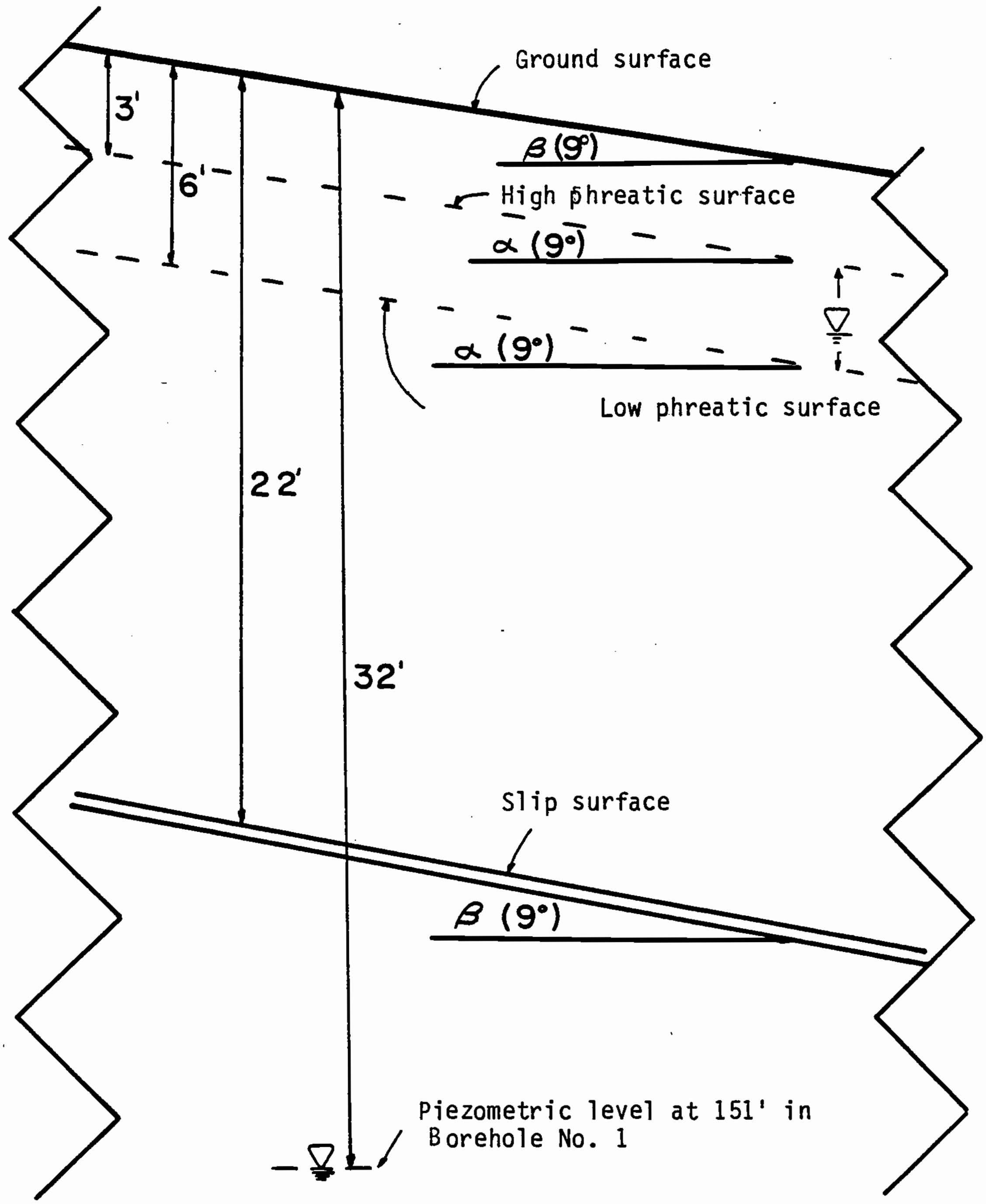

Figure 19. Generalized groundwater conditions in the Lookout Creek Earthflow.

loss is proportional to the length of the flow path and the hydraulic gradient, i, as described by Darcy's Law and the Bernouli Equation. The average slope gradient of the Lookout Creek Earthflow is 9°, and if flow were parallel to this slope angle, the seepage angle, α (Figure 19) would also be 9°. Although there must be many local variations in direction of groundwater flow in slope (due to the heterogeneous nature of the subsurface deposits) the average seepage angle between the Piezometers 1 to 3 should be around 9°.

In fairly uniform soils, such as those in an earth dam, it is often possible to predict pore water pressure based on theoretical applications of laws governing flow through porous media (Cedergren, 1977). Use of flow nets, or graphical solutions to the Laplace Equation applied to Darcy's Law, are of little use in understanding flow through heterogenous slides similar to the Lookout Creek Earthflow. The only way to determine the true nature of seepage forces and pore water pressures within a landslide is to use the results of field instrumentation (Ter-Stepanian, 1971). One of the best ways to do this is to use many carefully spaced piezometers. At each location, piezometers should be installed so that it is possible to determine piezometric level for at least two depths. Then it is possible to determine variations in pore water pressure and seepage forces throughout the depth of interest.

The groundwater information used in the subsequent engineering analysis of the Lookout Creek Earthflow was based on data from only three piezometers. Unfortunately, the piezometers measured pore water pressure at only one depth below the ground surface, so it was not possible to quantify the seepage angle at any point. Although the average seepage angle is probably close to 9°, for analysis pore pressures were determined using an average seepage angle varying between -10° and 20° (Figure 19).

Without actual measurement of pore pressures within the zone of failure or along the slip surface, it is difficult to determine the effective stress at any given time. Remoulding of clays, which may result from shearing along a slide boundary, can reduce soil permeability by one to four times (Mitchell, 1976). A change in the phreatic surface in much of a slope due to precipitation from storm events may not result in a significant change in pore pressure within a clay layer (Tavenas and Lerouiel, 1981). The major change in effective stress, due to increased pore pressure, is a function of the consolidation and permeability properties of the soil, and depending on these properties it may take several weeks for a pore pressure change to occur in a thick clay zone of very low permeability.

Movement

A major reason some landslides move rapidly and others move very slowly is related to the stress-strain properties of the soils in the failure zone. Highly overconsolidated and sensitive clays fail abruptly when stressed to beyond peak strength. Slides in these soils tend to be faster moving. On the other hand, if a slide soil has a perfectly plastic stress-strain curve there is little tendency for the slide to accelerate.

Many earthflows are found in old landslide debris (Skempton and Hutchinson, 1969). They may move on an old slip surface, after a change in slope geometry, hydrology, or soil weathering has altered the hillslope forces to the point where movement-causing forces are greater than movement resisting forces. After an initial failure, the strength of the slip surface will be very close to the residual strength. Post failure movements have been measured from near 0 to 600 cm per year, and are especially common in highly overconsolidated clays (Simmons and Menzies, 1978). However, because of the thixotropic nature of soils (tendency to change particle orientation and shear strength at constant volume), and factors such as weathering and changes in geometry, among others, shearing resistance will not be constant with time.

Based on the crackmeter data correlated with the stake array survey, measured yearly surficial movement has varied from 0.1 in. in the 1976-1977 water year, to 10 in. in the 1981-1982 water year. Measurable movement usually begins in November or early December, typically about three weeks after the first heavy fall rains. For the 5 years where there are reliable data, the piezometric level within the slide mass at the time movement begins is lower than the level during some of the time in the three weeks before movement started.

Downslope velocity of the slide was determined from the daily crackmeter data, as was the approximate daily acceleration. The values of velocity and acceleration are based on the interpretation of field crackmeter separation data, which are affected by machine characteristics, animals, and climatic events, especially snow. With these limitations considered, slide velocity at crackmeter 2 varies from non-measurable to 0.16 in/day $\pm$ 0.04 in/day. The acceleration of the Lookout Creek Earthflow varies from about -0.0013 in/day^2 to +0.0010 in/day^2.

There is variation in surface movement rates over the study area. The upper portion of the Lookout Creek Earthflow, for the two years of cross-section data, is moving downslope at a more rapid rate than the middle portion. This is probably related to the slide width; which is also the smallest in the upper portion. In the 1981-82 water year, the movement of the upper cross-section, A, was about 10 in., and that of the lower cross-sections was about 7.5 in. The movement recorded at crackmeter 2 is very similar to that determined in transverse stake section B, as both are in the central portion of the moving mass.

The block in the southwest corner bounded by crackmeter 1 has been moving faster than the rest of this slide, at least for the past four years where data are available. Based on data from stake arrays and crackmeter No. 1, this lower block moves between 0.5 in. and 2.0 in. per year faster than the main body of this mass-movement.

Movement velocity during the winter months appears somewhat related to piezometric levels, but there are many exceptions. Daily average velocity and piezometric levels are shown in Appendix A. Average piezometric level in the 1981-82 water year is nearly the same as the 1982-83 water year, yet yearly movement in 1981-82 was between 3 and 4 times greater than 1982-83. The major climatic difference between these two years was that in 1981-82 much of the precipitation came early in the year, and much of it was in the form of snow. The winter of 1982-83 was much warmer, and precipitation was distributed more evenly throughout the winter, with no large individual major storms.

External Factors Causing Movement

There are many external factors involved in causing mass movements. Slope movement is a response to these factors, and it is an oversimplification to state one external cause as being responsible for a given mass-movement. This study was concentrated on those factors which varied over the time where there is reliable field instrumentation data. Based on this information, some conclusions can be drawn as to the effect management activities will have on the slide if these activities can be seen as similar to the observed factors.

Those factors acting gradually over a long time scale, or those which act at very infrequent intervals, are very difficult to predict or evaluate quantitatively. Some of the factors which may act gradually to affect the stability of the Lookout Creek Earthflow include the following:

1. A decrease in slope gradient with time,
2. Internal weathering of the slope material changing both the engineering and physical properties, and
3. Erosion, especially due to Lookout Creek.
4. Changes in surface and subsurface drainage.

Because it was not possible to accurately quantify the factors listed above, they were not considered directly in the analysis.

Observed Factors

The directly observed factors which affect movement of the study area are those which cause fluctuations in the stresses in the shear zone at relatively constant material strength and slope geometry. The major short term factor is, therefore, the variability in the phreatic surface, and the associated change in pore water pressures and effective stress in the shear zone.

Another observed factor which could influence stability is the surcharge due to snow loading. The snow load is highly variable throughout the months from November through April, and was not directly measured during any studies of this earthflow (Harr, 1983). Therefore, snow load was assumed to vary from 0 ft. to 10 ft. of snow with 2 1/2 ft. of water equivalency, or a surcharge between 0 and 160 psf. Actual snow load is unlikely to reach this upper value. Although snow load was not measured directly, maximum snow load between

1980 and 1983 was determined based on a comparison between water input and precipitation data. The maximum surcharge from snow load was estimated to be around 30 psf between late December 1981 and February 1982; snow surcharge loads at other times were generally much smaller.

The effects of natural periodic events, such as major flooding, have an immediate effect on overall stability, but were not observed in this study. Periodic events were not considered in stability analysis, because quantitative relationships are very unpredictable.

Factors Affected by Forest Management

Management activities which may influence the stability and movement of the Lookout Creek Earthflow include changes in surcharge loading from vegetation and snow, and construction related changes in slope geometry. A 1 1/2 lane gravel road between 250 ft. and 400 ft. from the slide toe was constructed before 1960. Based on visual observation, the cut and fill slopes for this road were small (less than 10 ft.) and well balanced. The effect of this road on overall slope stability was fairly small because of the insignificant change in overall slope geometry.

The effect of vegetation directly on the movement of this slide is fairly minimal. In general, coniferous species in the study area have shallow rooting systems, and because the water table is always high, vegetation will not have a tendency to establish deep roots. Therefore, there should be no shearing resistance due to root strength along the basal slip surface. The shearing resistance due to roots along the edges of the slide must be insignificant, considering the extremely small area involved.

If the weight of vegetation on the slope is considered to be a surcharge load, removal of trees by forest harvesting may be considered a reduction in surcharge load. The total above ground biomass (weight of live and dead plant matter at watershed 10 in the Andrews Forest (Figure 1) varies from 220-435 tons/acre (Grier and Logan, 1977). The amount likely to be removed by harvesting is between 40% and 80% of the living tree stem wood, or between 110 to 330 ton/acre.

This is equivalent to a surcharge load of 5 to 15 psf. This surcharge is less than 1% of the soil pressure acting on the failure surface assuming an effective stress of about 1500 psf at 22 ft., therefore effects of vegetation surcharge removal are difficult to calculate with precision but will likely be insignificant. The volume of standing timber on the earthflow may be even less than that found at Watershed 10 because it is likely that there are fewer standing trees, as many have fallen. The weight of fallen trees and detritus may be nearly as much as standing tree stem weight, but little would be removed during logging operations.

The relationship between timber harvesting and snow accumulation is not completely clear. There are some indications that snow accumulation increases after harvesting, but there are no quantitative relationships useful for stability analysis available (Harr, 1983).

Vegetation also affects the hydrologic cycle. Water is removed from the forest environment by the evapotranspiration process (water returned to the atmosphere by evaporation and plant transpiration). This process is most active between April and October, though there is some evapotranspiration in the fall and winter months.

Studies in the H. J. Andrews Forest found average July evapotranspiration of about 4.0 in. of water, and average January evapotranspiration of about 0.25 in. (Rothacher and others, 1967). Evapotranspiration rates were reduced by a factor of three in summer months and two in winter months after forest clearcutting in North Carolina (Waring and others, 1981). On the Lookout Creek Earthflow, where winters are colder and wetter, clearcutting will have even less effect on winter evapotransiration rates.

The major effects of evapotranspiration on groundwater levels will take place in the spring and summer months. After clearcutting, water input to the soil should be increased. With increased water input, the groundwater level in a typical western Oregon slope should be higher, especially in the summer and at the start of fall rains. These higher groundwater levels will tend to reduce slope stability in a typical western Oregon hillslope.

Stability Analysis Procedures

Methods of slope stability analysis include limit equilibrium procedures, finite element methods, limit analysis, discrete particle analysis, and rheological analysis. Limit analysis and discrete particle methods are presently in the phase of theoretical development (Chen, 1969, Hodge, 1976). Limit analysis, discrete particle, were not used to analyze the Lookout Creek Earthflow.

In the finite element method, the slope mass is modeled as an array of very small, discrete elements, usually with stress strain properties based on elastic, or elasto-plastic theories. Finite element procedures are used in many applications other than slope analysis. With detailed soil property and slope geometry data, finite element procedures can be used to determine certain stress-strain relations and locations of stress concentrations within the slope. However, with the finite element method, it is difficult to evaluate large strains similar to those around the shear zone of the Lookout Creek Earthflow. Therefore, the finite element method was not used in this study.

Limit Equilibrium

At the present time, the Limit Equilibrium Method is the most commonly used slope stability analysis procedure. Limit Equilibrium procedures are based on perfectly plastic behavior along a failure surface, and not the actual stress-strain behavior of soils. Failure of a slope mass is assumed to occur simultaneously at every point on the failure surface. The entire slide or failure mass is considered to be a rigid body. In limit equilibrium methods, slopes are evaluated with respect to the factor of safety, which is a ratio of forces resisting failure to forces causing failure.

For the Lookout Creek Earthflow, the infinite slope method of stability analysis is the least complicated procedure applicable. In this procedure, the ground surface is assumed to have a uniform slope, extending infinitely upslope and downslope at the same inclination. All soil strata must be inclined at the slope surface angle, and extend infinitely, and surcharge loads must be uniform over the entire slope. Although no natural slopes are true infinite slopes, for practical purposes in slides with roughly planar failure surfaces, the greater the length to depth ratio the more reasonable the infinite slope assumption.

A typical section from an infinite slope is shown in Figure 20. The factor of safety for an infinite slope, FS, in a c'-ϕ' soil with seepage parallel to the slope and a surcharge is given by

$$FS = \frac{c' + (S + \gamma h - \gamma_w d) \cos^2 \beta \tan \phi'}{(\gamma h + S)(\cos \beta \sin \beta)}$$

where:

C' is the effective cohesion

S is the surcharge loading

γ' is the total soil unit weight

h is the depth from ground surface to slip surface

γ_w is the unit weight of water

d is the depth from phreatic surface to the slip surface

β is the slope inclination

ϕ' is the effective angle of internal friction

In this equation, total saturated and non saturated unit weights are assumed to be equal.

For slides with widely varying depths to the failure surface, a two-dimensional non-circular method of analysis will generally be more accurate than the infinite slope procedure. Analysis of circular slip surfaces with 2-dimensional methods is generally more straight-forward than analysis of non-circular surfaces. However, non-circular slip surfaces (Figure 21) are more commonly found in actual slides (Morgernstern and Price, 1965). For the Lookout Creek Earthflow, where the location of the slip surface is either known or inferred, non-circular analysis should be more precise than circular analysis.

The SSTAB1 slope stability computer program was used for engineering analysis of the Lookout Creek Earthflow (Wright, 1974). This program may be used for circular and non-circular slip surfaces, though only the non-circular procedures were used in this study. A basic assumption in the SSTAB1 program is that resultant side forces on each wedge are parallel. With this assumption, all conditions for static equilibrium of the forces on the idealized slope are met. The factor of safety for the slope, with given geometry, soil properties, and piezometric line, is found by successive iterations of the equilibrium equation which has the factor of safety term on both sides.

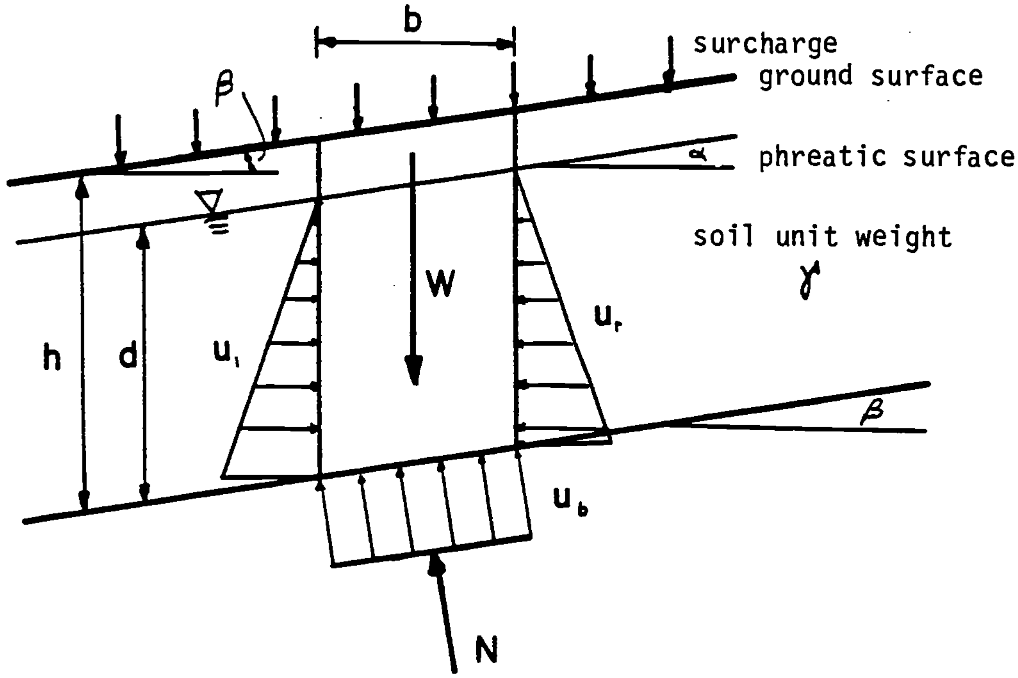

Figure 20. Section of an infinite slope in $c'-\phi'$ soil with seepage.

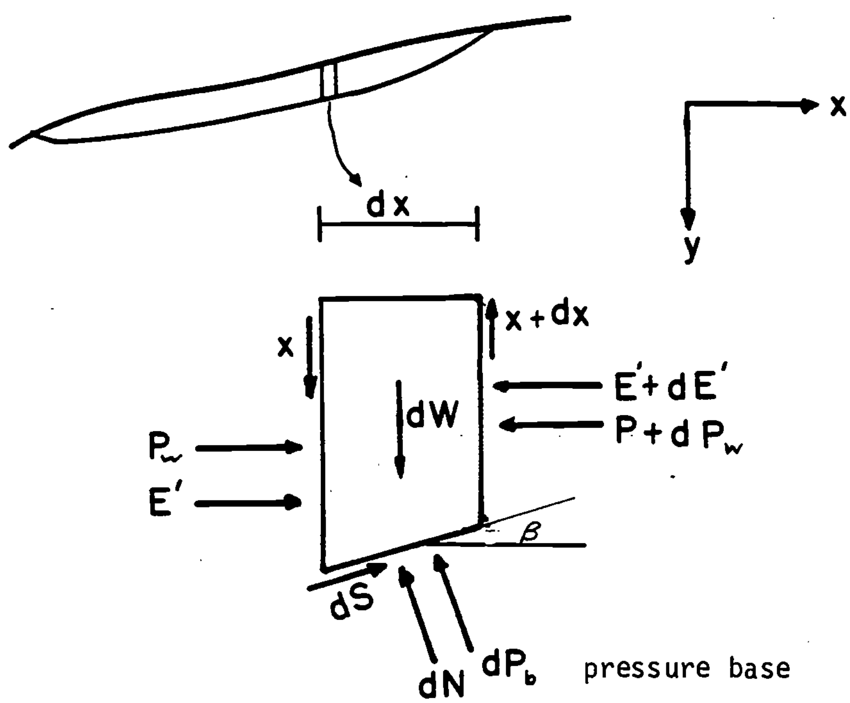

Figure 21. Forces acting on wedge for 2-D Non-circular Analysis (after Morgenstern and Price, 1965).

Backanalysis

In the typical engineering slope stability analysis, the unknown parameter is the factor of safety. Backanalysis for slope stability calculations is a procedure often used to determine other slope parameters when the factor of safety is known. For existing slides, the factor of safety was very close to 1.00 when slide movement started. It is possible to calculate one average strength parameter from back analysis of slope stability based on pre-slide slope geometry and groundwater conditions. Or, with strength parameters and initial slope geometry conditions known, it is possible to calculate average groundwater conditions when movement began.

Three-Dimensional Effects

For slides and earthflows with very complex geometries, a three-dimensional limit equilibrium procedure may be useful (Hovland, 1978). Three-dimensional effects have been found to be 1) more significant for smaller lengths of the failure mass, 2) important on gentle slopes with soils having high cohesion interept and low friction angle, 3) such that, under certain circumstances in cohesionless soils the 3-D factor of safety is less than the 2-D, and 4) the ordinary 2-D method of slices generally yields quite satisfactory results for most slopes (Chen and Chameau, 1983). The only justification for use of 3-D analysis is the availability of very reliable groundwater, subsurface geometry, and material property data. This was not the case for the Lookout Creek Earthflow. Also, because the

Lookout Creek Earthflow is long and slender, and soil cohesion is probably very small, three-dimensional effects are probably insignificant. No three dimensional analyses were employed in this study.

Rheological Analysis

Rheology is the study of deformation and flow of matter. Most rheological models are developed for a specific type of mass-movement, and are useful in describing average deformations. For an analysis of movement rates and the nature of deformation, a rheological analysis may be necessary. However, rheological calculations often require many soil and slope parameters which are difficult to evaluate.

The depth creep of slopes has sometimes been referred to as a slow earthflow type movement (Ter-Stepanian, 1963). Models have been developed to describe the deformation and velocity of a soil layer undergoing creep (Yen, 1969, Savage and Chleborad, 1982). These models are based on the assumptions that: 1) the soil behaves as a rigid visco-plastic material, 2) the soil is at residual shearing strength, and 3) the material is isotropic and at constant volume.

Rheological analysis based on the Yen and Ter-Stepanian models was attempted as part of the engineering analyses in this study. With the velocity of the Lookout Creek Earthflow known, and residual strength parameters known, the coefficient of viscosity was back calculated. However, movement rates are highly variable with only minor stress changes. For a rheological model to describe this variation, the coefficient of viscosity would also have to vary. Thickness of the creeping layer was back-calculated using a coefficient of viscosity, η , equal to 2 x 10^{10} lb. sec./ft.^{2}, an average value found by Yen (1969). Thickness determined from these calculations varied between 8 ft. and 20 ft., which did not correlate with much smaller values found later in this paper.

Limitations of Analysis Procedures

There are many assumptions required for all slope analysis procedures, and some of these were covered in the preceding sections. However, most of these assumptions are related to differences between the analysis procedure and some ideal slope. There are still further limitations involved when applying these procedures to natural slopes, especially to active mass-movements. These additional limitations may be summarized as follows.

1. The movement of the Lookout Creek Earthflow is not a a dynamic process, and as such this movement is governed by kinematic physical laws. The slide was considered to be a rigid body in all considerations of velocity and acceleration.
2. Even though different portions of the Lookout Creek Earthflow move at different rates, the slide was considered as a continuum for engineering analysis. The effects of extensional and compressional movement

have been investigated, but the effects on overall slope stability are not clear (Keefer, 1978).

3. The lack of sufficient field data made the consideration of progressive failure (Bjerrum, 1967) unfeasible. To evaluate progressive failure, subsurface stress conditions must be better defined, in order to identify areas of stress concentration.

VI. RESULTS

Shear Zone Characteristics

The depth to, and thickness of the shear zone were determined based on subsurface deformation of the tube installed in bore hole number 1. This tube, 1" inside diameter, was installed to a depth of 151 ft. In May 1983, after 5 months, a 4" long rod passed to the bottom of the tube, while a 10" rod stopped at a depth of 21.5 feet. It was assumed that a 7" (mid length between the 4" and 10" rods) rod would just pass by the zone of deformation in the tube.

Thickness of the deformed zone was calculated assuming uniform circular curvature of the tube, as shown in Figure 22. The surface downslope movement at borehole 1 was 2.15 in. (5 1/2 cm), and displacement at the bottom of the shear zone was assumed to be zero. With a 7 inch long, 1/2" diameter rod just touching the sides of the tube, the zone of deformation is approximately 10 inches thick, as shown in Figure 22.

This is an approximate determination of slide zone thickness. Thickness of the shear zone may also vary throughout the slope. It is also possible that other shear zones exist below this one. However, the 4 inch rod did pass to the bottom of the tube, and the recorded piezometric level in the tube indicates that there are no breaks. The existance of multiple shear zones is therefore unlikely. At borehole 1 the depth to the center of this shear zone is 21.5 ft.

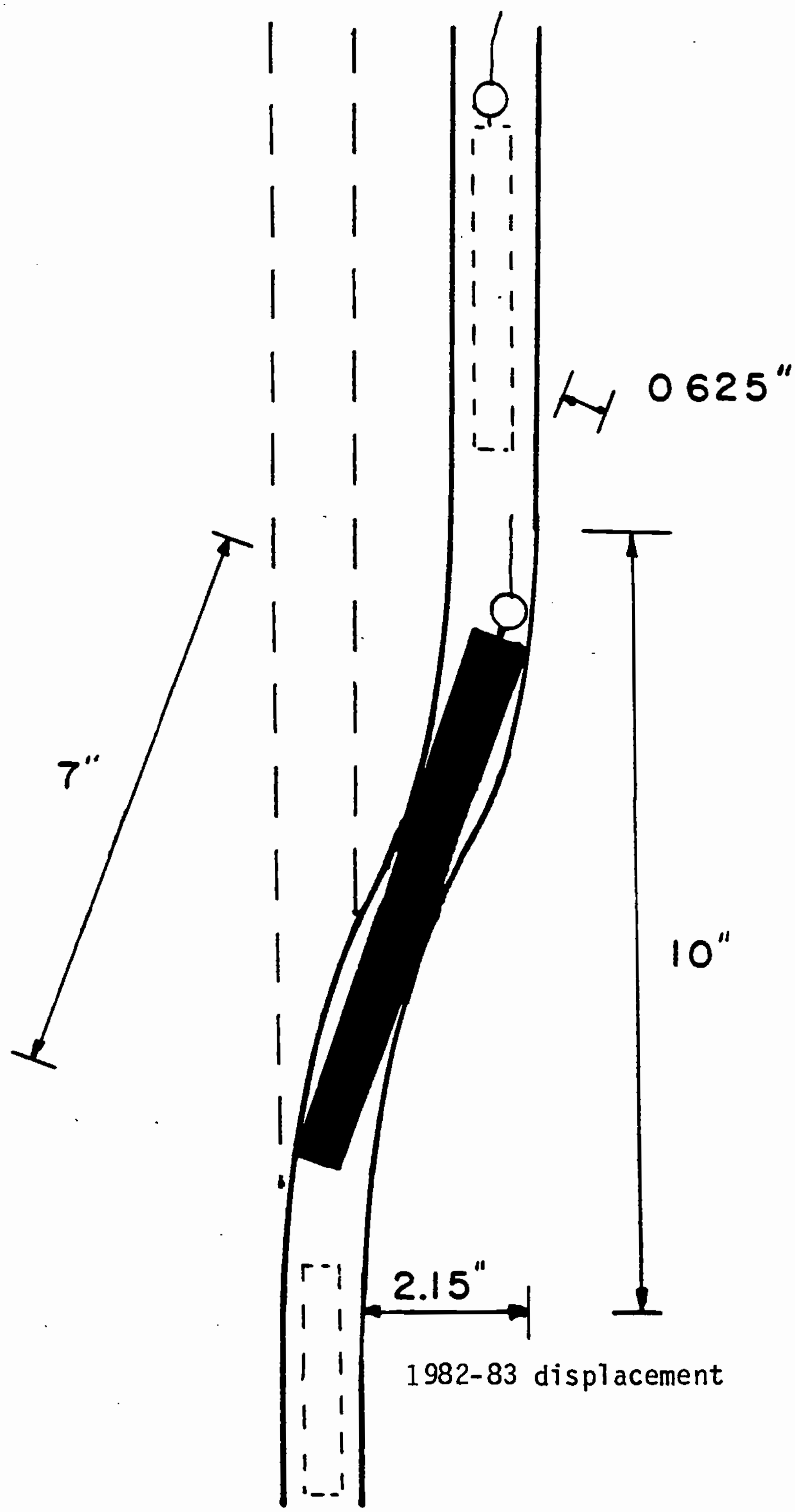

Figure 22. Determination of shear zone thickness.

Mass Flow and Continuity

Volume flux of this slide was determined using the surficial movement record from transverse section C. Velocity was assumed to be constant with depth in the slide mass. Data from both inclinometer and PVC tube deformation indicate virtually all deformation takes place in about a ten inch thick shear zone. The depth to failure surface at borehole 1, on transverse section C, was found to be 21.5 ft. The average elevation of the slide along section C is equal to the elevation at the top of borehole 1, so the average depth to slip surface along cross-section C was also assumed to be 21.5 feet. Yearly volume movement was equal to the area under the surficial movement versus cross sectional distance curve (Figure 1) multiplied by the average depth (21.5 ft). Calculated volume flux was 200 yd^3 for the 1981-82 water year, and 55 yd^3 for the 1982-83 water year.

The principle of continuity is used to determine average depth to failure plane at transverse sections A and B. Continuity is based on the conservation of mass, which states that the mass within a system stays constant with time

$$dm/dt = 0$$

where m is the system mass, and t is time.

In slides where there is significant extension and compression, continuity would not be a reasonable assumption.

Longitudinal profile measurements on this slide were inconclusive because of insufficient precision, but this indicates that extension and compression were not large relative to the absolute value of recorded movements. The average depth to the slip surface was calculated using the 1981-82 volume movement and dividing by the area of the cross-section displacement plots (Figure 9). Depths to slip surface for both upper sections was 25 $\pm$ 1', for years 1981-82, and 1982-83. Depths to slip surface for the rest of the slide was based on interpolation and extrapolation, as shown in Figure 23.

Evaluation of Field Investigation

One of the objectives of this study was to evaluate the field investigation procedures used on the Lookout Creek Earthflow. The surface deformation, subsurface deformation, and groundwater instrumentation methods, as well as subsurface investigation and sampling techniques, are evaluated on the following pages. The procedures are evaluated with respect to: necessity for analysis, reliability, precision, simplicity, cost, and nature of measurement or sampling process.

The surface deformation instrumentation used on the Lookout Creek Earthflow included: crackmeters, transverse stake sections, a longitudinal stake section, and stake arrays. The subsurface deformation measuring devises were inclinometers and a PVC tube, and groundwater was monitored with open tube and recording piezometers. The instrumentation is evaluated as follows:

1. Benefit for analysis,

   ++ of much use in analysis

   \+ of use in analysis

   \- of little use for analysis

   -- useless for analysis

2. Reliability of data from instrumentation,

   ++ data from one set of measurements relate directly to movement mechanics

   \+ data from several measurements may be used to determine movement mechanics

   \- data from several measurements can be useful in understanding general relationships

   -- unreliable data

3. Ease of data interpretation,

   ++ can determine parameter directly from one simple measurement

   \+ Determination of parameter with simple calculations

   \- Determination of parameter usually requires detailed understanding of instrumentation

   -- Determination of parameters indirect or requiring complex mathematical solution

4. Precision of instrumentation for measuring, groundwater levels or deformations,

   ++ to nearest .01 cm or better

   \+ to nearest 0.1 cm or better

   \- to nearest 1.0 cm or better

   -- less accurate than 1.0 cm

5. Cost of instrumentation (average cost for 1 set-up for five years, plus fixed costs),*

   $ less than $250

   $$ between $250 and $1000

   $$$ more than $1000

6. Measurements by periodic surveys (P), or continuously recording instrumentation (C).

Table IV evaluates these parameters for the instrumentation procedures used on the Lookout Creek Earthflow.

The subsurface investigation methods are evaluated in Table V. Drilled boreholes, deep test pits, surface sample pits are evaluated as follows:

1. Determination of subsurface conditions for analysis,

   ++ straightforward interpretation of subsurface conditions.

---

*Cost of subsurface instrument allocated between sampling and instrumentation.

+ Investigation leads to significant understanding of subsurface conditions

- Possible to interpret certain subsurface properties.

-- Difficult or impossible to interpret subsurface conditions in a landslide

2. Reliability of data from investigation,

++ relatively undisturbed samples from the shear zone can be obtained

+ disturbed samples can be obtained and undisturbed samples may possibly be obtained from around the shear zone

- Disturbed samples may be obtained from the shear zone, or undisturbed samples may be obtained from other than the shear zone

-- samples are not obtained

3. Cost of a single borehole, pit, or one seismic line (some relative values as instrumentation),

4. Ease of installation of subsurface instrumentation,

++ Instrumentation can be installed which measures undisturbed field conditions

+ Both piezometers and inclinometers may be installed, though data may not represent undisturbed conditions

Table IV

Evaluation of Lookout Creek Earthflow Instrumentation

	Benefit for Analysis	Relia-bility	Ease of Data Interpretation	Pre-cision of Measure-ments	Cost	Continuous/Periodic
Surface Deformation Instrumentation						
Crackmeter	+	-	-	-	$$	C
Transverse stake section	++	+	+	++	$	P
Longitudinal stake section	+	-	-	--	$$	P
Stake Array	-	+	+	-	$	P
Subsurface Instrumentation						
Open tube piezometer	-	+	+	-	$$	P
Recording piezometer	++	-	-	+	$$$	C
Inclinometer	++	-	-	++	$$$	P
1" PVC tube	+	+	+	--	$$	P

Table V

Evaluation of Exploration Techniques

	Determination of Subsurface Conditions for Analysis	Reliability for Sampling	Cost	Installation of Instrumentation
Borehole	+	+	$ $ $	+
Deep test pit	-	+	$ $	-
Surface sample pit	--	-	$	--
Geophysical methods	--	--	$	--

- Instrumentation may be installed, but data is unlikely to represent field conditions

-- Instrumentation can not be installed.

There are many other types of instrumentation, that, although not used in this study, might be of use in further studies of the Lookout Creek Earthflow or similar mass movements. Included with these are extensiometers to measure large subsurface deformations, and transducers for determination of pore water pressures in a shear zone (Fukuoka, 1980, Wilson and Mikkelsen, 1978).

Stability Analysis

Using infinite slope analysis, determination of strength parameters based on F.S. = 1 is rather straightforward. A factor of safety of 1.00 for a moving earthflow is a reasonable assumption (VanOine, 1980).

The equation for the factor of safety of an infinite slope with no surcharge and seepage parallel to the slope is:

$$FS = \frac{c + (\gamma h - \gamma_w' d) \cos^2 \beta \tan \phi'}{\gamma h \cos \beta \sin \beta}$$

For a residual soil, the cohesion can be assumed to be zero. When movement begins, the factor of safety is 1.00, so the above equation may be arranged to determine the residual angle of internal friction,

$$\phi'_r = \tan^{-1} \left[\frac{\gamma' h \tan}{\gamma' h - \gamma'_w d}\right]$$

where parameters are shown in Figure 20, as previously described.

The average parameters used for backanalysis were:

γ = 120 lb./ft^3 = (1.92 g/cm^3), (total soil unit weight)

h = 22 ft. = (6.7 m), (average depth, ground surface to slip surface)

β = 9° (average inclination of slope)

γ_w = 62.4 lb./ft^3 = (1.00 g/cm^3), (unit weight of water)

d = 16 ft. to 19 ft. (4.9 m to 5.8 m), (depth, phreatic surface to slip surface).

Soil unit weight, γ, was based on laboratory test data and typical density values of the flow rocks. The height of the phreatic surface (groundwater table) above the slip surface was based on maximum and minimum yearly values from the piezometers.

The actual water level, d, when the slope begins to move was not completely clear, so backanalysis was used with FS = 1 for both the highest and lowest conditions measured. For the high water situation, ϕ_r equals 16.0° at FS = 1, and for the lowest phreatic condition ϕ_r was calculated to be 14.3°. Actual ϕ_r must be between these two values. Assuming actual ϕ_r is 15°, the factor of safety at high water is 0.93, and the factor of safety at lowest water is 1.05. Therefore, high water factor of safety is about 11% less than low water factor of safety.

The SSTAB computer program was used in the 2-dimensional stability analysis of this earthflow (Wright, 1974). A total of 15 slope coordinate points each were used to describe surface geometry, location of phreatic surface, and location of the slip surface, as shown

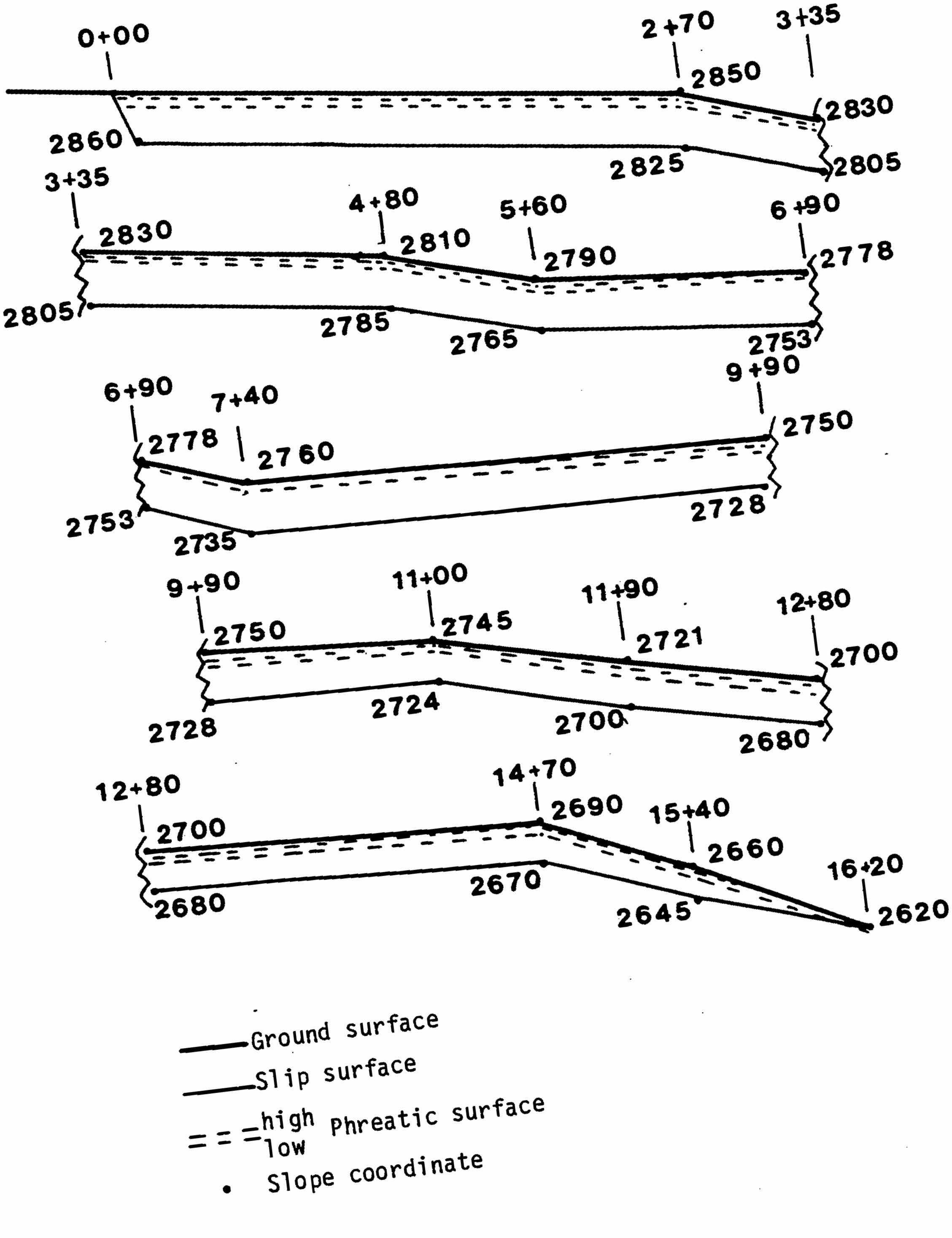

Figure 23. Slope configuration used in 2-D analysis.

in Figure 23. The location of the slip surface and phreatic surface are based on limited and/or indirect data and calculations, so the 2-dimensional slope analyzed here may not best represent actual field conditions.

Backanalysis was conducted assuming $c_r = 0$ and FS = 1. For the highest pheatic level, ϕ'_r was determined to be 15.8 degrees, and with the lowest phreatic level was determined to be 13.9 degrees. These values are about 2 percent lower than those determined with infinite slope analysis, partly because infinite slope procedures do not consider the additional shearing resistance in the head and toe areas of the slide. Overall, the results are very similar, and therefore the infinite slope technique was used for most of the following comparisons because of its simplicity.

<u>Lab Test and Backanalysis Comparison</u>

The correlation between shear strength parameters obtained from laboratory strength tests and slope stability backanalysis was very poor. Therefore, soils tested from the Lookout Creek Earthflow toe are very different from the soils in the shear zone. Why there is a layer of very weak soils at the base of this mass movement is difficult to explain with the available evidence. The differences between shear zone soils and those tested for strength parameters probably include some basic compositional differences. After sample shearing back and forth in the direct shear test, only a slight drop in angle of internal friction was observed, so the soils in the shear zone are not equivalent to tested soils which have been sheared to the residual strength.

Seepage Angle

The seepage angle, α (Figure 20) was varied to see what the effect would be on slope stability. Backanalysis was performed with $c' = 0$, and average α equal to -10°, 0°, 9°, 20°. The phreatic surface was assumed to be constant, at 5 feet below the ground surface. Except for the 9° angle, the other angles were used to average represent groundwater conditions for complicated subsurface flow, and not actual conditions. The factor of safety equation for c and s equal to zero is as follows:

$$FS = \frac{(\gamma' h \cos^2\beta - \gamma'_w d \cos^2\alpha) \tan\phi'}{(\gamma' h \cos\beta \sin\beta)}$$

with $\alpha = \beta = 9^\circ$, h = 22' and d = 18', backanalysis gives $\phi' = 15.4$ for F.S. = 1.00. Then with $\phi = 15.4^\circ$, the factor of safety for the other seepage angles is as follows:

-10°: FS = 1.00

0°: FS = 0.98

20°: FS = 1.07.

Therefore, the effect of different seepage angles is fairly insignificant, and using $\alpha = \beta$ is reasonable because it reflects the average conditions over the length of the slide. The purpose of this investigation was to compare stability for different stress conditions. For this purpose exact values either of shear strength parameters or groundwater conditions were not essential. On a steep slope, or where there is upward seepage, the direction of seepage flow has a much greater influence on stability. The deep piezometer in borehole 1 indicates that there is a perched water table within the Lookout Creek Earthflow as shown in Figure 19. Therefore, upward seepage is unlikely, and the use of a 9° seepage angle is reasonable.

Surcharge

Snow and vegetation were considered to be uniform surcharge loads which might be influenced by forest management activities. Maximum snow surcharge is probably around 160 psf, though the largest calculated snow surcharge between 1980 and 1983 was 30 psf. If the trees on the Lookout Creek were harvested by clearcutting, this would be equivalent to a surcharge reduction of about 5 psf.

The effects of surcharge on overall stability in the study area was considered only with the infinite slope procedure. Backanalysis was performed first with $c_r = 0$ and surcharge equal to 0, assuming a 3 ft. depth to phreatic surface. Using the value of ϕ_r' from the backanalysis, surcharges were added, and the corresponding factor of safety computer. As the surcharge increased, the factor of safety increases slightly.

Factor of safety with surcharge was determined using data from backanalysis with no surcharge and factor of safety equal to 1.00. For the factor of safety to be decreased by an increased surcharge, the cohesion must be greater than about 150 psf, and ϕ_r less than around 9°. If the cohesion along the slip surface is fairly high, then an increased surcharge could reduce overally stability. However, available evidence leads to the conclusion that cohesion is near zero, as described in the next section. For soils with low cohesion, backanalysis shows vary little effect of surcharge on factor of safety.

One possible factor which might influence stability is non-uniform snow load. The earthflow is located in the transient snow zone. Snow distribution over the slope may vary with local topography, vegetation, and elevation. Variation of surcharge load, and effects of forest operations, are dependent on these and many other factors. To analyze effects of non uniform surcharge, detailed field data are necessary. Since these data were not available, and also because surcharge variations are likely to be very small, the effect of surcharge variation on slope stability was not considered quantitatively in this study.

Evapotranspiration

Removal of vegetation by forest practices, including logging and slash burning, is likely to result in reduced evapotranspiration from the Lookout Creek Earthflow. The greatest reduction will take place in the spring and summere, when plants are more actively growing and temperatures are warmer. In the fall and winter, evapotranspiration will be substantially reduced, but not eliminated. Evapotranspiration rates are not being measured on the Lookout Creek Earthflow.

Within two days of a large winter storm, groundwater level at Piezometer no. 1 will typically rise to within 3 or 4 ft. of the ground surface. If there is little or no rainfall for several weeks after such a large storm, groundwater drops to near low summer values. During mid-November 1980 and late January 1981, groundwater levels were within 4 ft. of the ground surface. In late November 1980 and early February 1981, after 2 to 3 week periods with little or no precipitation, groundwater levels dropped to 5.75 ft. Lowest summer groundwater level in 1981 or 1982, after 2 or 3 months with little rainfall, was about 6 ft. below ground surface. These nearly equal values of groundwater level, when evapotranspiration differs by a factor of about 10, indicate that summer evapotranspiration does not seem to have a major effect on groundwater levels in the Lookout Creek Earthflow. Given this, changes in evapotranspiration that result from clearcutting should have little effect on groundwater levels as well.

Other factors associated with clearcutting may also have an effect on groundwater levels in the Lookout Creek Earthflow. Overland flow may result because of soil compaction or the presence of fire related hydrophobic soils. It is uncertain what the magnitude of these effects might be.

Acceleration

Acceleration rates of the Lookout Creek Earthflow were calculated from the crackmeter displacement record. The forces acting on a

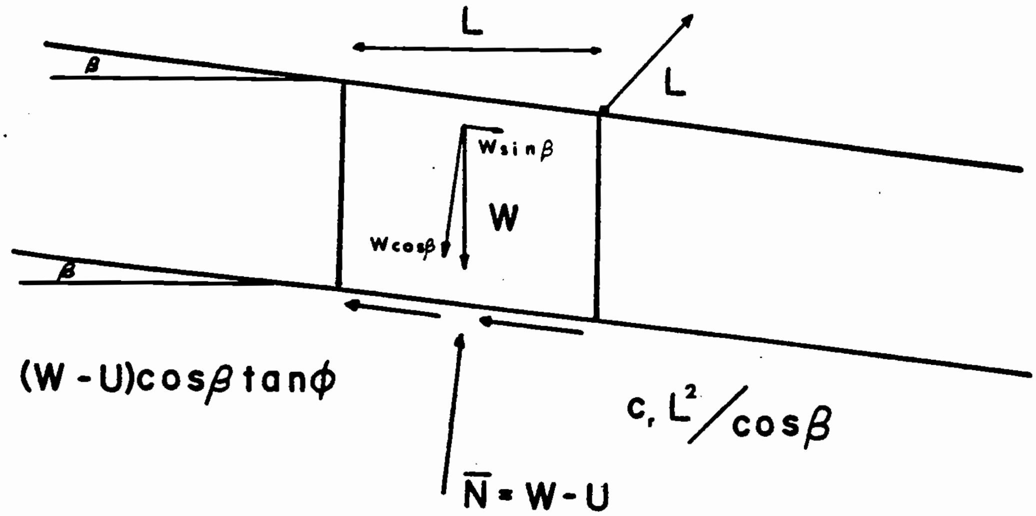

Figure 24. Forces for acceleration.

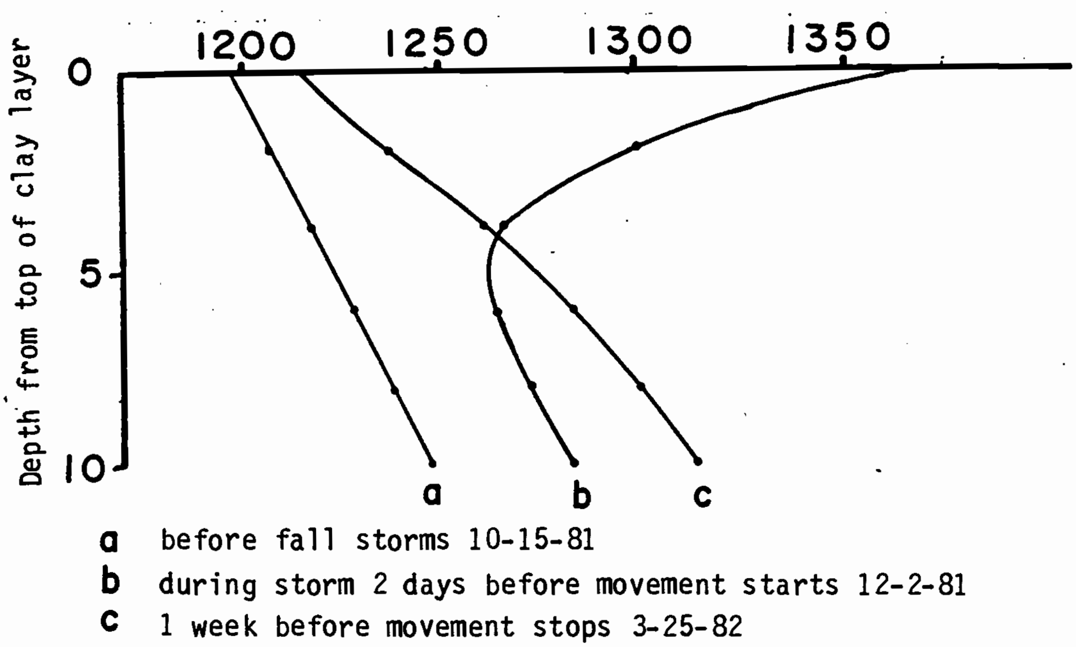

Figure 25. Pore pressure variations determined in clay layer determined by finite difference technique.

section of this slide are shown in Figure 24. Assuming this to be a rigid body section of an infinite slope, Newtons Second Law, F = ma, can be applied as follows:

$$ma = -W\sin\beta + C_r/\cos\beta + (W-u)\cos\beta\tan\phi_r'$$

where

m = mass of the section (W/g)

W = weight of section

β = inclination of the slope

c_r - the residual cohesion

u - average pore pressure on the base

ϕ_r' - the residual angle of internal friction

This equation was rearranged to calculate change in pore pressure associated with a given acceleration as follows:

$$u = \frac{-(c_r/\cos\beta) + \gamma' h\left(\frac{a}{g} + \sin\beta - \cos\beta\tan\phi'_r\right)}{\cos\beta\tan\phi'_r}$$

When the maximum acceleration determined from the Crackmeter 2 record, 0.4 mm/day^2 (4.4 x 10^{-13} ft/sec^2) is used in the above equation, the effect on pore pressure is extremely negligible.

In a soil at residual strength, further shearing should not result in volume change and associated pore pressure change. The results from the triaxial testing, even though not representative (in terms of shear strength) show that the A_f pore pressure parameter is fairly close to 0. This is indicative of little volume change during shearing. Therefore, it is assumed that there is very little or no shear generated pore water pressure change of soils in the Lookout Creek Earthflow.

Consolidation

Measurable movement of the Lookout Creek Earthflow generally begins and often continues when the recorded piezometric levels are lower than they had been during periods of no movement. Therefore, the rate process of volume change due to increased pore pressures with relatively constant total stress was examined. The consolidation process is usually considered to be the volume change associated with increased total stress, and the time dependent increase in effective stress and decrease in pore pressure. Any change in effective stress within a soil must be accompanied by a change in volume of the soil, and for saturated soils the rate of volume change is controlled by the permeablety and consolidation characteristics of the soil.

To illustrate the effect that this process could have on the Lookout Creek Earthflow, the slope was considered to be a highly permeable material above a relatively impermeable shear zone. With this assumption, pore pressures within the shear zone can be determined from using daily measured piezometric levels adjusted to the top of the shear surface. A finite difference solution technique was used to determine pore pressures in a 10" (thickness determined by tube deformation) clay layer. Inflow and outflow of water in response to changing phreatic level was assumed only across the upper surface of the shear zone.

The parameters required for this analysis included the layer thickness, coefficient of consolidation, saturated and moist unit weight, thickness of the soil profile above the clay layer, and initial distance to the piezometric surface above the shear zone. The average thickness of the clay layer was assumed to be ten inches, the thickness of the zone of deformation determined from the tube in borehole 1. The coefficient of consolidation used for analysis, which relates layer thickness and time to the percent of total volume change, was 0.01 ft^2/day (0.001 in^2/min). This value is the minimum value determined from triaxial and consolidation testing, about 25% of the average c_v determined in the tests. The saturated and moist unit weights used for analysis were 120 and 115 pounds per cubic foot, respectively. Thickness of soil profile was assumed to be 25', and initial distance to piezometric surface was 18.86 feet, based on data from Piezometer number 1.

The variation of pore pressure with depth at various times is shown in Figure 25. There is a clear relationship between start of movement and pore water pressure calculated at the center of the shear zone. At this time, pore water pressure at the center of the shear zone is between 45 and 55 psf greater than pressure at the start of fall rains. The exact pressure varies depending on local variations in depth from shear zone to initial phreatic surface and ground surface. The maximum observed pore water pressure increase was 90 psf in February, 1982. The difference between the pore pressure required to initiate movement and the maximum observed pore pressure is 40 psf. This is only 3 percent of the initial pore water pressure (1150 psf) associated with average initial groundwater conditions. Results from this finite difference solution are shown, along with corresponding movement rates, in Figures 26 to 28.

Rate of movement is influenced by many factors other than pore water pressure at one point in the center of the shear zone, including variations in groundwater levels, shear zone thickness, transient loads, non-uniform movement rates, and many other factors. Nonetheless, there appears to be a relationship between crackmeter determined movement rates and pore water pressures determined from consolidation theory. As pore pressure increases there appears to be an increase in observed movement. However, a quantitative relationship to predict movement rate based only on excess pore pressure would be of little or no use, because of all the other factors involved. Even slight changes in the long-term factors may be of significance because of the very slight stress changes associated with movement.

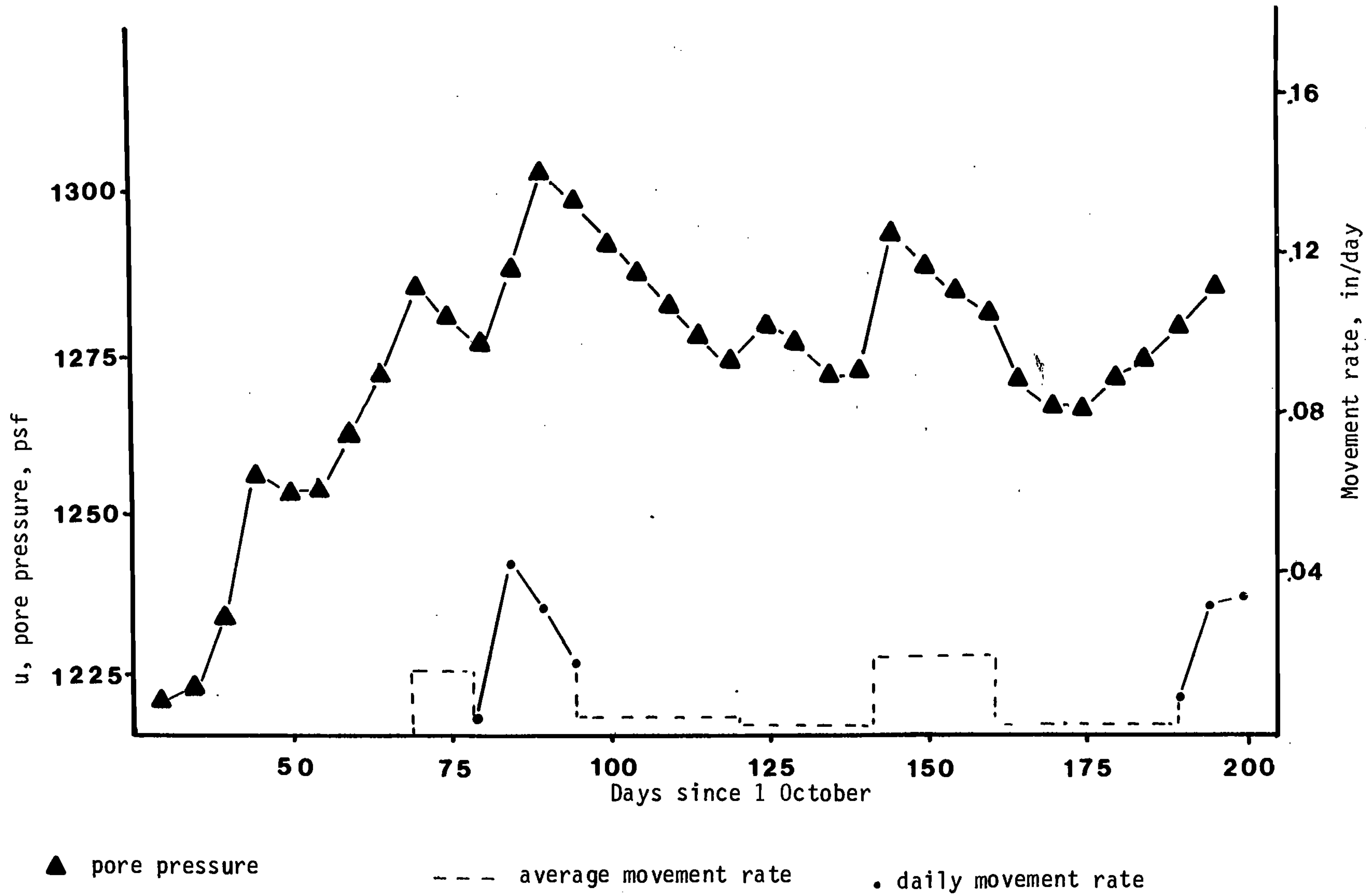

Figure 26. Pore pressure at center of clay layer and movement rates, 1980-81.

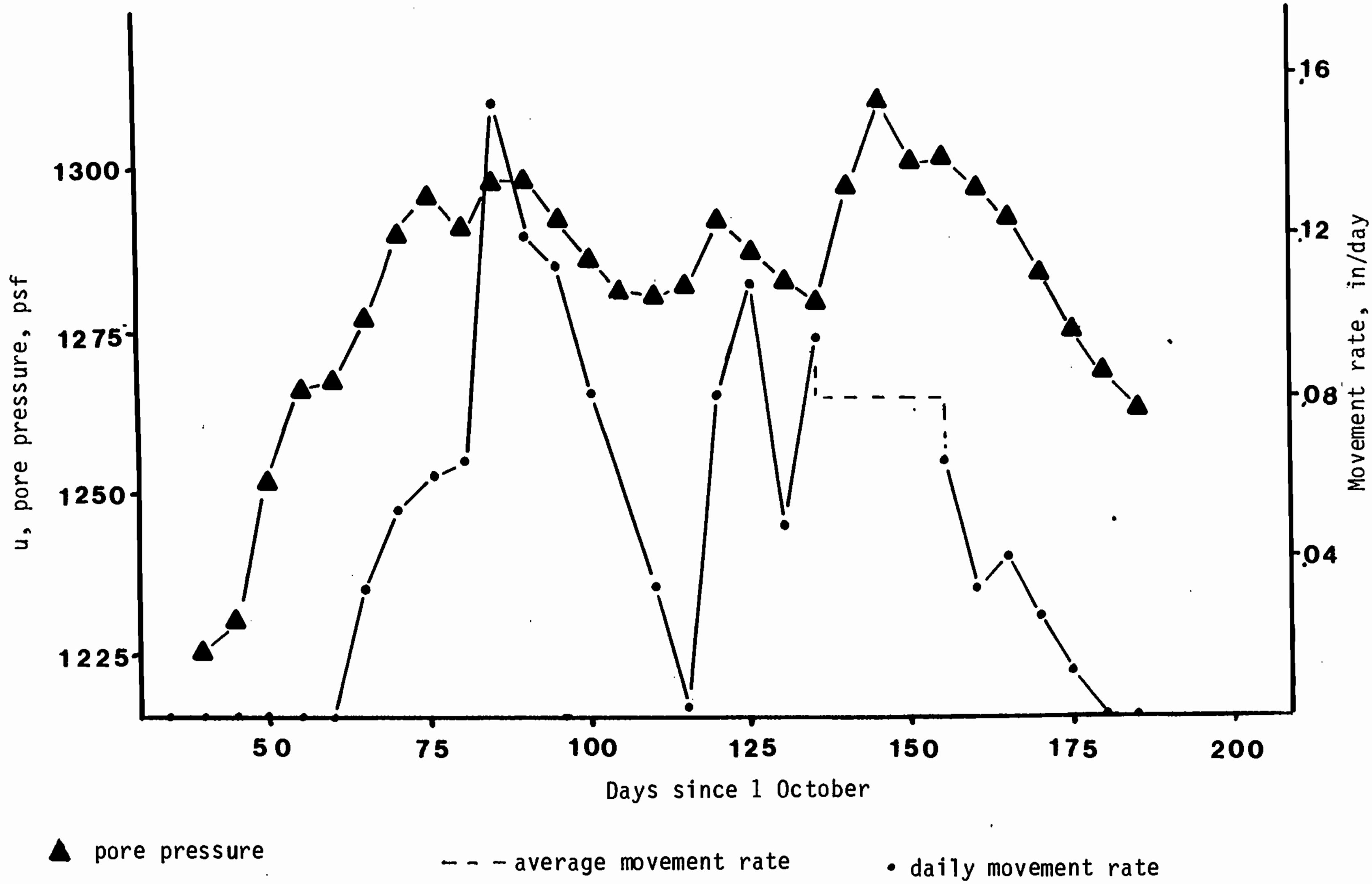

Figure 27. Pore pressure at center of clay layer and movement rates, 1981-82.

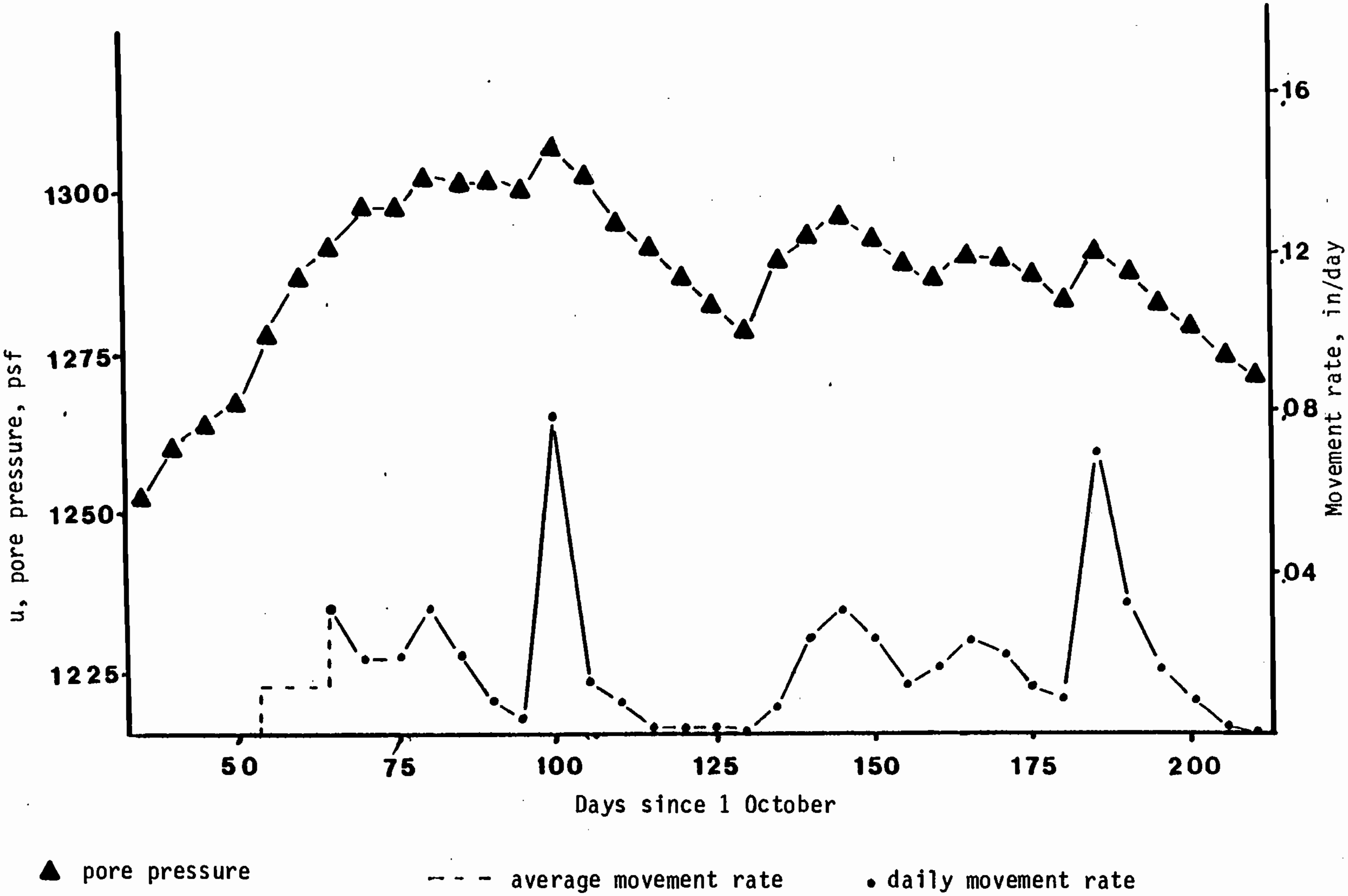

Figure 28. Pore pressure at center of clay layer and movement rates, 1982-83.

VII. CONCLUSIONS

General

The movement of the Lookout Creek Earthflow is better explained by shear failure along a definite boundary than by depth creep of the slope. Rheological creep models can not account for the variations in movement rate associated with minimal stress changes. The rate of movement, the number of years this feature has moved, and the nature of surface and subsurface soils, indicate that at least some of the soils in the Lookout Creek Earthflow have a complex geomorphic history.

The soils in the zone of deformation are at, or very close to, the residual shear strength. The measured accelerations are very small, and slopes with soils at residual strength have little tendency to accelerate. Acceleration in such a slope would be caused by a major slope alteration and change in stresses, not by a reduction in strength of soil in the shear zone due to stress concentrations or large strains. In a soil existing at residual strength the cohesion should be small, or non-existant, and behavior should be close to that of a purely-frictional soil. Slope backanalyses also indicate little or no cohesion, because a cohesive soil would not have the same response to increased pore water pressure.

The timing of movement is directly related to the volume change process in the shear zone soils. This is quite evident in Figures 26 to 28. This volume change process is dependent on piezometric changes associated with rainstorms and snowmelt, and soil thickness, permeability, and consolidation properties. When the pore water pressure in the shear zone increases about 45-55 psf above typical summer values, movement begins. This increase is equal to the pressure associated with about 0.8 feet of water, much less than the three feet change observed in the piezometers above the shear zone. There is some relationship between movement rate and pore water pressure in the center of the shear zone, though the maximum pressure increase above summer levels is only about 80 psf, or 1.3 feet of water. With such a slight change in pore pressure, many other factors have a significant effect on movement rates.

There is so much variation among features defined as "earthflows" that review of literature on these landslides did not result in much information useful for engineering analysis of the Lookout Creek Earthflow. Because of the complex geomorphic nature of the study area, field reconnaissance and instrumentation were essential for engineering analysis of the slide. However, because of these complexities, even more field instrumentation and more subsurface investigation, are required for understanding the geologic history and more about the mechanics of this slide. Samples are required directly from the failure zone of the Lookout Creek Earthflow to quantify properties relevant to the movement of this slide. Soils tested from the toe area were not representative of soils with properties similar to those determined from backanalysis. Also, if the soil properties determined from backanalysis, and groundwater conditions, were applied to slopes adjacent to the earthflow, most of these other areas would have a factor of safety less than 1.0, as the adjacent ground, for the most part, is steeper. However, these adjacent slopes are stable.

Effects of Forest Operations

In general, most typical forest management operations should have little effect on the movement characteristics of the Lookout Creek Earthflow. The snow and vegetation loads can be modeled as fairly uniform surcharge loads. As such, in an almost purely frictional (residual shear strength) soil, minor changes in snow and vegetation loading should have little effect on overall slope stability. After a timber harvest, the existing "surcharge" on the Lookout Creek Earthflow would be reduced. However, in the winter months, there may be an increased snow load on the slope after a clearcut, due to reduced sublimation potential from vegetation and other factors. Overall average change in surcharge is very difficult or impossible to determine, so quantitative prediction of movement change after clearcutting is not feasible. However, present surcharge variation appears to have little effect on overall slope stability, hence, movement changes should be small.

The subsurface flow characteristics of the Lookout Creek Earthflow are different from a typical slope in the Oregon Cascades. Clearcutting and/or burning of forest vegetation should reduce water loss by evapotranspiration. However, evapotranspiration effects do not seem to have a significant impact on groundwater levels in the Lookout Creek Earthflow. Any changes in timing and magnitude of movement resulting from timber harvesting are not expected to be significant. The subsurface hydrology of the Lookout Creek Earthflow is not similar to that of stable slopes of western Oregon, and may not be representative of other "earthflow" mass-movements. These conclusions on forest management effects are reasonably valid only for the Lookout Creek Earthflow.

The effects of road building were not evaluated quantitatively. The existing road caused such slight changes in slope geometry that, with the uncertanties in all the other data, stability with and without the road could not be reasonably evaluated.

Suggestions for Further Research

Much of the engineering analysis of the Lookout Creek Earthflow was based on indirect observations, calculations with little data, and many assumptions. To further investigate the mechanics of this mass-movement, additional instrumentation is required. Determination of actual physical and engineering properties of the soils in the shear zone mandates direct sampling of these soils.

To verify and quantify the volume change phenomenon in the shear zone, well sealed piezometers are required in this zone. Piezometers which might withstand the deformation of the shear zone would probably be electric type transducers. A study which examines the groundwater conditions determined from many well spaced piezometers in relation to slope stability factor of safety was reported by Kenney and Chan (1977), and provides valuable guidelines.

The nature of deformation, strain, and strain rate of the soils within the shear zone probably requires instrumentation with extensiometers (Fukuoka, 1980). Inclinometers installed on the Lookout Creek Earthflow failed to penetrate the shear zone, so were of little use. Even if installed through the shear zone, the tube may be sheared off and determination of shear zone deformation may not be possible. For quantification of movement characteristics of this and similar mass soil movements, fairly detailed subsurface instrumentation is essential.

BIBLIOGRAPHY

Akroyd, T. 1957. Testing in Soil Engineering. Marshall Press, London.

American Geological Institute. 1976. Dictionary of Geological Terms. Anchor Press. Gorden City, N.Y.

Atkinson, J.H. 1981. Foundations and Slopes. McGraw-Hill Book Co., U.K. Maindenhead, England.

Bishop, A.W. and L. Bjerrum. 1960. The Relevance of the Triaxial Test to the Solution of Stability Problems. Proc. Research Conf. on the Shear Strength of Cohesive Soils, Boulder, Colorado. pp. 437-501.

Birkeland, P.W. 1974. Pedology, Weathering, and Geomorphological Research. Oxford University Press, London.

Bjerrum, Laurits. 1967. Progressive Failure in Slopes of Overconsolidated Plastic Clay and Clay Shales. Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM5, pp. 3-49.

Bowles, J.E. 1978. Engineering Properties of Soils and their Measurement. McGraw-Hill, New York.

Cedergren, H. 1977. Seepage, Drainage, and Flow Nets. J. Wiley and Sons, New York.

Chen, R.H. and J.H. Chameau. 1983. Three Dimensional Limit Equilibrium Analysis of Slopes. Geotechnique, Vol. 32, No. 1, pp. 31-40.

Chen, W.F. 1969. Soil Mechanics and Theorems of Limit Analysis. Journal of Soil mechanics and Foundations Division, ASCE, Vol. 95, No. SM2, pp. 493-518.

Chowdhury, R.N. 1978. Slope Analysis. Elsevier Pub. Co., London.

Crandell, D.R. 1965. The Glacial History of Western Washington and Oregon, In: Quarternary of the United States. Wright, H.E. ed., Princeton University Press, Princeton, N.J.

Filz, G. 1982. Engineering Properties of Southeast Alaskan Forest Soils. M.S. Thesis. Oregon State University.

Fukuoka, M. 1980. Instrumentation: Its Role in Landslide Prediction and Control. Proc. Int. Symposium on Landslides. New Delhi, India, pp. 139-153.

Gottesfeld, A.S., F.J. Swanson, and L.M. Gottesfeld. 1981. A Pleistocene Low-Elevation Subalpine Forest in the Western Cascades, Oregon. Northwest Science, Vol. 55, No. 3, pp. 157-167.

Gray, D.H. 1977. Creep Movement and Soil Moisture Stress in Forests vs. Cut-over clopes. Dept. of C.E., Univ. of Michigan. Unpublished report no. 74-02427.

Grier, C.C. and R.S. Logan. 1977. Old Growth Pseudotsuga menziesii communities of a Western Oregon Watershed: Biomass Distribution and Production Budgets. Ecological Monographs, Vol. 47, pp. 373-400.

Harr, R.D. 1981. Some Characteristics and Consequences of Snowmelt During Rainfall in Western Oregon. Journal of Hydrology, Vol. 53, pp. 277-304.

Harr, R.D. 1983. Personal Communication.

Hodge, R.A. 1976. Regional Geology, Flow Systems, and Slope Stability. M.A. Sc. Thesis, Dept. of Geol. Sci., Univ. of British Columbia, Vancouver.

Hovland, H.J. 1977. Three-Dimensional Slope Stability Method. Journal of the Geotechnical Engr. Div., ASCE, Vol. 103, No. 6T9, pp. 971-986.

Keefer, D.K. 1977. Earthflow. PhD. Thesis, Dept. of Applied Earth Sciences, Stanford Univ., Stanford, Cal.

Kelsey, H.M. 1978. Earthflows in Franciscan Melange, Van Duzen River Basin, California. Geology, Vol. 6, pp. 361-364.

Kenney, T.C. and H.T. Chan. 1977. Seasonal changes in groundwater pressure and stability of a natural slope in varved clay. Dept. of C.E., University of Toronto Pub. No. 77-01.

Komamura, F. and R.J. Huang. 1974. New Rheological Model for Soil Behavior. Journal of the Geotechnical Division, ASCE, Vol. 100, No. GT7, pp. 807-824.

Lefebvre, G. 1981. Fourth Canadian Geotechnical Colloquium: Strength and Slope Stability in Canadian Soft Clay Deposits. Canadian Geotechnical Journal, Vol. 18, No. 3, pp. 420-442.

Lowe, J., and P.F. Zaccheo. 1975. Subsurface Explorations and Sampling, In: Foundation Engineering Handbook. H.F. Winterkorn and H.Y. Fang, ed., Ch. 1 Van Norstrand Reinhold Co., New York.

Maeda, T., H. Takenaka, and B.P. Warkentin. 1977. Physical Properties of Allophane Soils. Advances in Agronomy, Vol. 29. pp. 229-264.

Mitchell, J.K. 1976. Fundamentals of Soil Behavior. J. Wiley and Sons, New York.

Morgenstern, N.R. and V.E. Price. 1965. The Analysis of the Stability of General Slip Surfaces. Geotechnique, Vol 15, No. 1, pp. 79-93.

Newmark, N.M. 1960. Failure Hypothesis for Soils. Proc. Research Conf. on the Shear Strength of Cohesive Soils, ASCE. Boulder, Colorado. pp. 17-32.

Parfitt, R.L. and T. Henmi. 1982. Comparison of an Oxalate-Extraction Method and an Infrared Spectroscopic Method for Determining Allophane in Soil Clays. Soil Sci. Plant Nutr., Vol. 28, No. 2, pp. 183-1900.

Peck, D.A., A.B. Griggs, H.G. Schlicker, F.G. Wells, and H.M. Dole. 1964. Geology of the Central and Northern Parts of the Western Cascade Range in Oregon. U.S.G.S. Professional Paper No. 449.

Rothacher, J., C.T. Dyrness, and R.L. Fredriksen. 1967. Hydrologic and Related Characteristics of Three Small Watersheds in the Oregon Cascades. U.S. Forest Service PNW Experiment Station Paper.

Savage, W.Z. and A.F. Chleborad. 1982. A Model for Creeping Flow in Landslides. Bulletin of the Assoc. of Engr. Geologists, Vol. XIX, No. 4, pp. 333-338.

Scott, W.E. 1977. Quarternary Glaciation and Volcanism, Metolius River Area, Oregon. Geological Society of America Bulletin, Vol. 88, pp. 113-124.

Simmons, N.E., and B.K. Menzies. 1978. The Long-term Stability of Cuttings and Natural Clay Slopes, In: Developments in Soil Mechanics - 1. C.R. Scott ed., Ch. 10, Applied Science Publishers, London.

Skempton, A.W. 1961. Effective Stress in Soils, Concrete, and Rock. In: Pore Pressure and Suction in Soils. Butterworth Pub. Co., London.

Skempton, A.W. and J. Hutchinson. 1969. Stability of Natural Slopes and Embankment Foundations. Proceedings of the 7th Int. Conf. on Soil Mechanics and Foundations, Mexico City, Vol. 3, pp. 291-340.

Sowers, G.F. and D.L. Royster. 1978. Field Instrumentation, In: Landslides, Analysis and Control, ch. 4. Transportation Research Board, Washington, D.C.

Spangler, M.G. and R.L. Handy. 1982. Soil Engineering. Harper and Row Co., New York.

Swanson, F.J., R.D. Harr, and R.L. Fredriksen. 1979. Geomorphology and Hydrology in the H.J. Andrews Experimental Forest, Western Cascades. USDA Forest Service PNW Research Station Field Trip Guide.

Swanson, F.J., and M.E. James. 1975. Geology and Geomorphology of the H.J. Andrews Experimental Forest, Western Cascades, Oregon. USDA Forest Service Research Paper PNW-188.

Swanson, F.J. and D.N. Swantson. 1977. Complex Mass-movement Terrains in the Western Cascade Range, Oregon. Reviews in Engineering Geology, Vol. III, pp. 113-124.

Swanston, D.N. and F.J. Swanson. 1976. Timber Harvesting, Mass Erosion, and Steepland Forest Geomorphology in the Pacific Northwest, In: Geomorphology and Engineering. Coates, D.R. ed., Dowden, Hutchinson and Ross, Inc. Stroudsburg, Pa., pp. 199-221.

Taskey, R.D. 1978. Relationship of Clay Mineralogy to Landscape Stability in Western Oregon. PhD. Thesis, Oregon State University.

Tavenas, F., and S. Leroueil. 1981. Creep and Failure of Slopes in Clays. Canadian Geotechnical Journal, Vol. 18, No. 1, pp. 106-120.

Ter-Stepanian, G. 1963. On the Long-term Stability of Slopes. Norwegian Geotechnical Institute Pub. 52, pp. 1-13.

Ter-Stepanian, G. 1971. Piezometric Method of Determination of Seepage Force in a Landslide Body. Proc. 4th Asian Regional Conf. of Soil mech. and Found. Engr., Bankok, Vol. 1, pp. 455-460.

Terzaghi, K. 1950. Mechanics of Landslides, In: Application of Geology to Engineering Practice, Berkey Volume. Geological Soc. Am., pp. 83-123.

Terzaghi, K. and R. Peck. 1967. Soil Mechanics in Engineering Practice. J. Wiley and Sons, New York.

Thayer, T.P. 1939. Geology of the Salem Hills and North Santiam River Basin, Oregon. Ore. Dept. of Geol. and Min. Ind. Bull. No. 15.

VanDine, D.F. 1980. Engineering Gelogy and Geotechnical Study of Drynoch Landslide, British Columbia. Geological Survey of Canada professional paper 79-31.

Varnes, D.J. 1978. Slope Movement Types and Processes, In: Landslides, Analysis and Control, Ch. 2. Transportation Research Board, Washington, D.C.

Waring, R.H., J.J. Rogers, and W.T. Swank. 1981. Water Relations and Hydrologic Cycles. In: Dynamic Properties of Forest Ecosystems, Ch. 4. D.E. Reichle, ed. Cambridge Univ. Press, Cambridge.

Williamson, D.N. 1964. Geology Report on the Blue River Saddle Sam. U.S. Army Corps of Engineers, Portland District, Foundations and Materials Branch Report.

Williamson, D.N. 1983. Personal Communication.

Wilson, S.D., and P.E. Mikkelsen. 1978. Field Instrumentation, In: Landslides, Analysis and Control, Ch. 5. Transportation Research Board, Washington, D.C.

Wright, S.G. 1974. SSTAB1 - A General Computer Program for Slope Stability Analysis. Dept. of Civil Engr., Univ. of Texas at Austin Research Report no. GE-74-1.

Woods, M.C. 1977. Ice Age Geomorphology, Middle Fork San Joaquin River, Madera County, California. California Geology, Vol. 30, No. 11, pp. 249-253.

Yen, B.C. 1969. Stability of Slopes Undergoing Creep Deformations. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 95, No. SM4, pp. 1075-1096.

Zaruba, Q. and V. Mencl. 1982. Landslides and Their Control. Elsevier Pub. Co., London.

APPENDICES

APPENDIX A

PRECIPITATION, TEMPERATURE, GROUNDWATER LEVEL AND MOVEMENT RECORDS

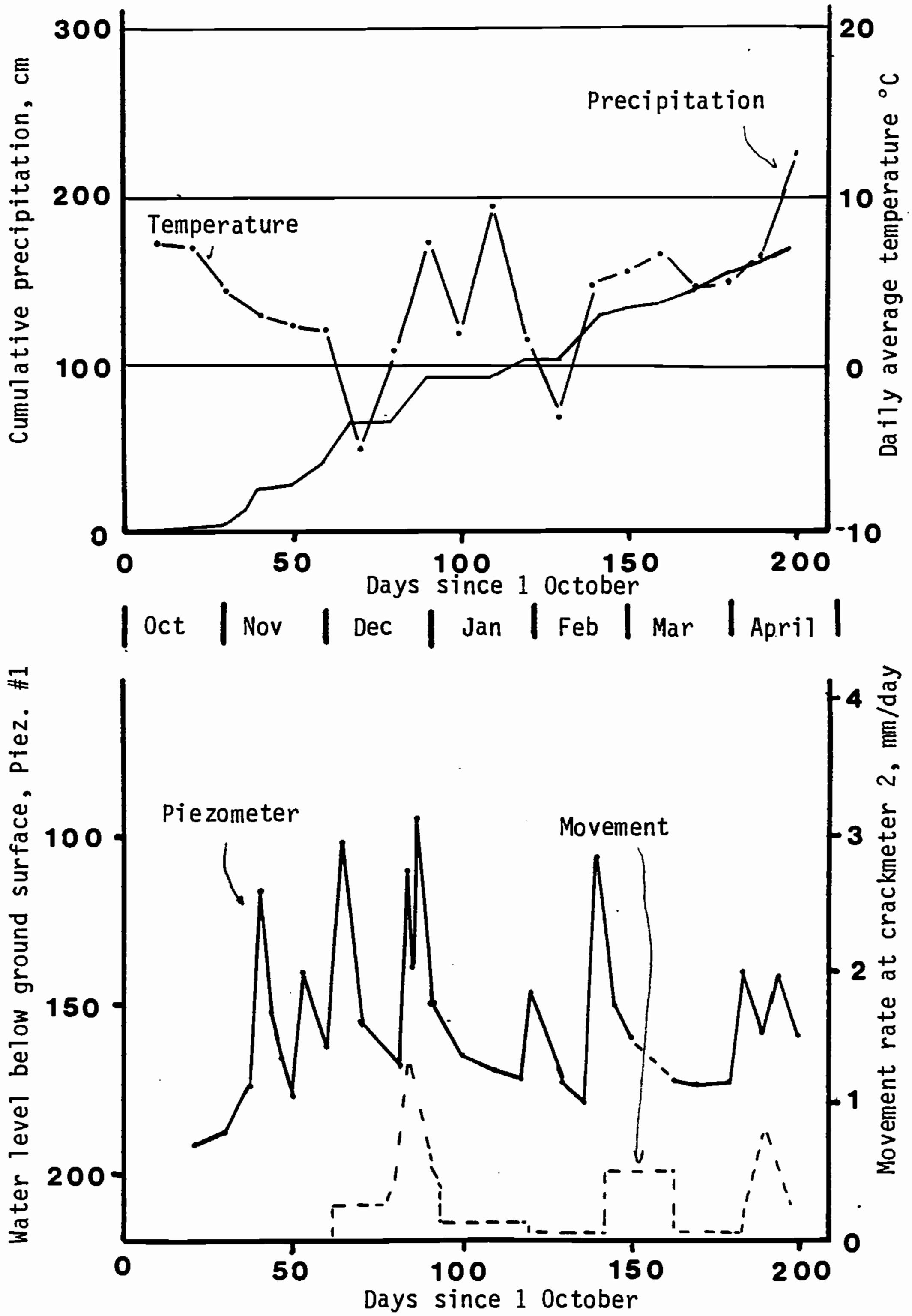

Figure 29. Precipitation, temperature, groundwater level and slide velocity for the Lookout Creek Earthflow, 1980-81.

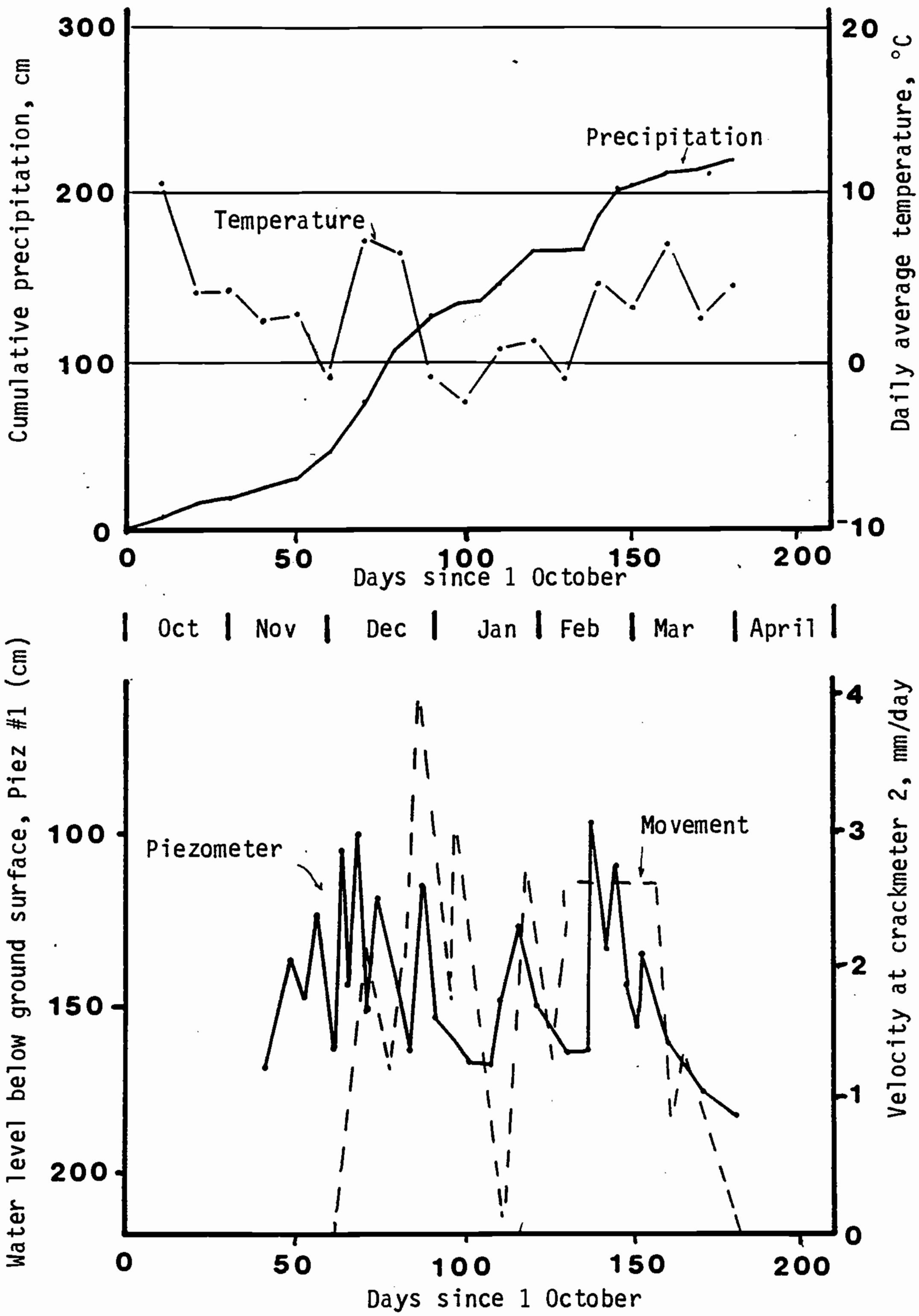

Figure 30. Precipitation, temperature, groundwater level, and Slide velocity for the Lookout Creek Earthflow, 1981-82.

APPENDIX B

BORE HOLE NUMBER 1 LOG

Lookout Creek Earthflow
Borehole Log No. 1
Drill Rig: Acker Mark IV
Hole Elevation 2745 ft.

H.J. Andrews Experimental Forest
12-13-82 to 12-15-82
Hole Diameter 3" NQ-wireline
Depth of Hole 151.2'

Elevations	Depth (feet)	Unified Class.	Material Description	Core Recov.	% Solids	Data
2745 2741		SM	brown gravelly silty sand (fill)	23	23	
	10	SM	gray to brown gravelly silty sand, stiff.	10	10	Oxidation of soil to 27º
2729				33	33	16º sample 25
	20	SM MH Fines	light brown gravelly silty sand with plastic fines	20	8	21^{5} failure zone
				7	6	27º sample 26
2718	30	SM, MH fines	gray still gravelly silty sand with plastic fines	60	34	
				24	22	
2707				24	4	
	40	SM, ML fines	brown medium stiff bouldery silty sand with fines of low plasticity	52	44	
				23	23	
				60	56	
	50			12	10	
				100	59	55º sample 27
	60			13	8	
				41	35	
2674	70			44	38	

Lookout Creek Earthflow Borehole Log No. 1 (cont.)

Elevations	Depth (feet)	Unified Class.	Material Description	Core Recov.	% Solids	Data
2674				39	37	
			Gray brown medium stiff bouldery silty sand with gray with gray igneous boulders	63	63	77^{8} wood fragment $C^{14} > 40\,000$ years
	80			22	22	
				48	48	
	90			5	5	
				56	56	
	100			44	44	
				33	33	
2639			brown stiff silty sand w/pyroclastic fragments.	20	2	106^{0} sample 29
2635	110		Gray stiff sandy silt w/igneous fragments.	43	35	
2630		GP	rounded gravel and small cobbles (alluvium)	2	2	
	120			11	11	
				5	5	
2614	130	GP	gray cobbles and boulders	26	26	
				100	100	
	140			71	71	
				24	24	
				47	47	
2593^{8}	150					151^{2} bottom of hole

APPENDIX C

RESULTS OF SOIL MINERALOGY TESTS

Table VI. Results from tests for Allophane clay content

	Oxalate μM/g			Oxalate g/g			%	15 bar water	15 bar water
	Al	Fe	Si	Al	Fe	Si	Carbon	airdry	moist
Sample 2 Toe site A	<75.6	554.5	42.1	<.0020	0.0310	0.0012	2.81	21.8	25.3
Sample 5 Toe site C	<98.8	<28.1	<28.0	<0.0027	<0.0016	<0.0008	<0.56	52.6	35.2
Sample 6 Toe site D	<2397.	<341.0	<339.2	<0.0647	<0.0190	<0.0095	<0.62	39.5	35.0
Sample 21 Surface debris flow	<325	<46.3	<46.0	<0.0088	<0.0026	<0.0013	<0.5	22.0	26.1
Sample 25, 16 ft. borehole 1	73.3	108.5	46.3	0.0020	0.0061	0.0013	<0.7	24.4	24.3

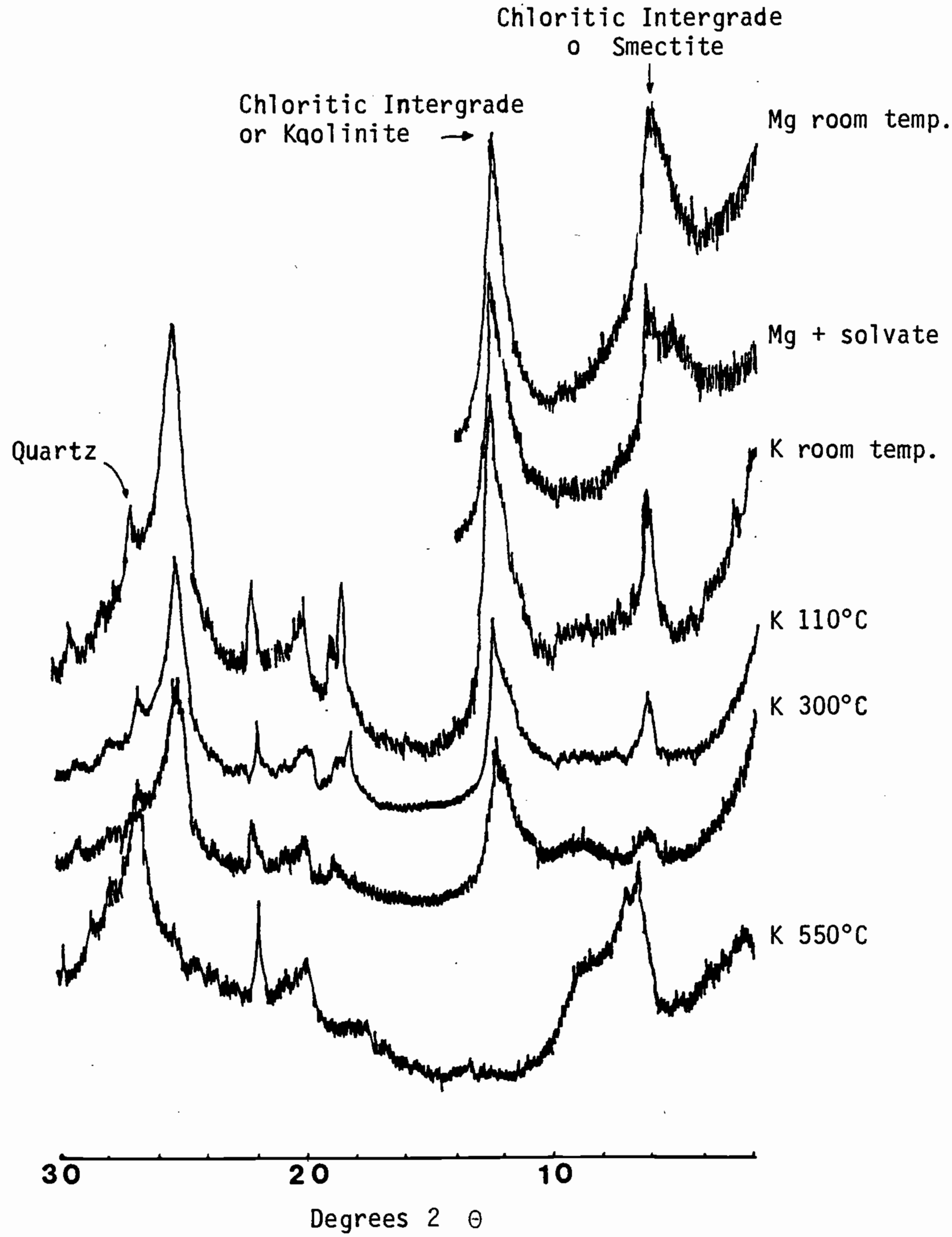

Figure 31. X-ray diffraction of gray MH silt from toe site A.

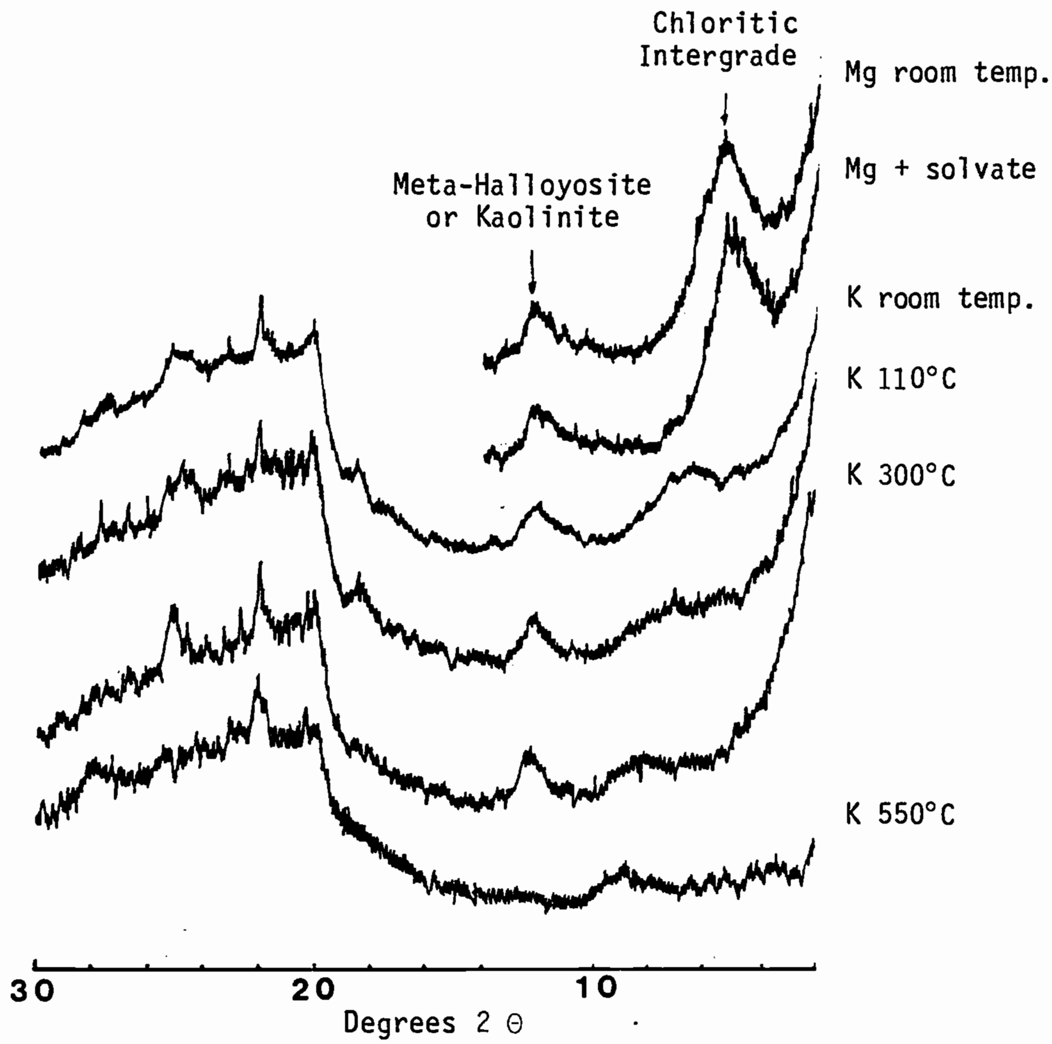

Figure 32. X-ray diffraction of mottled white MH silt, from toe site C.

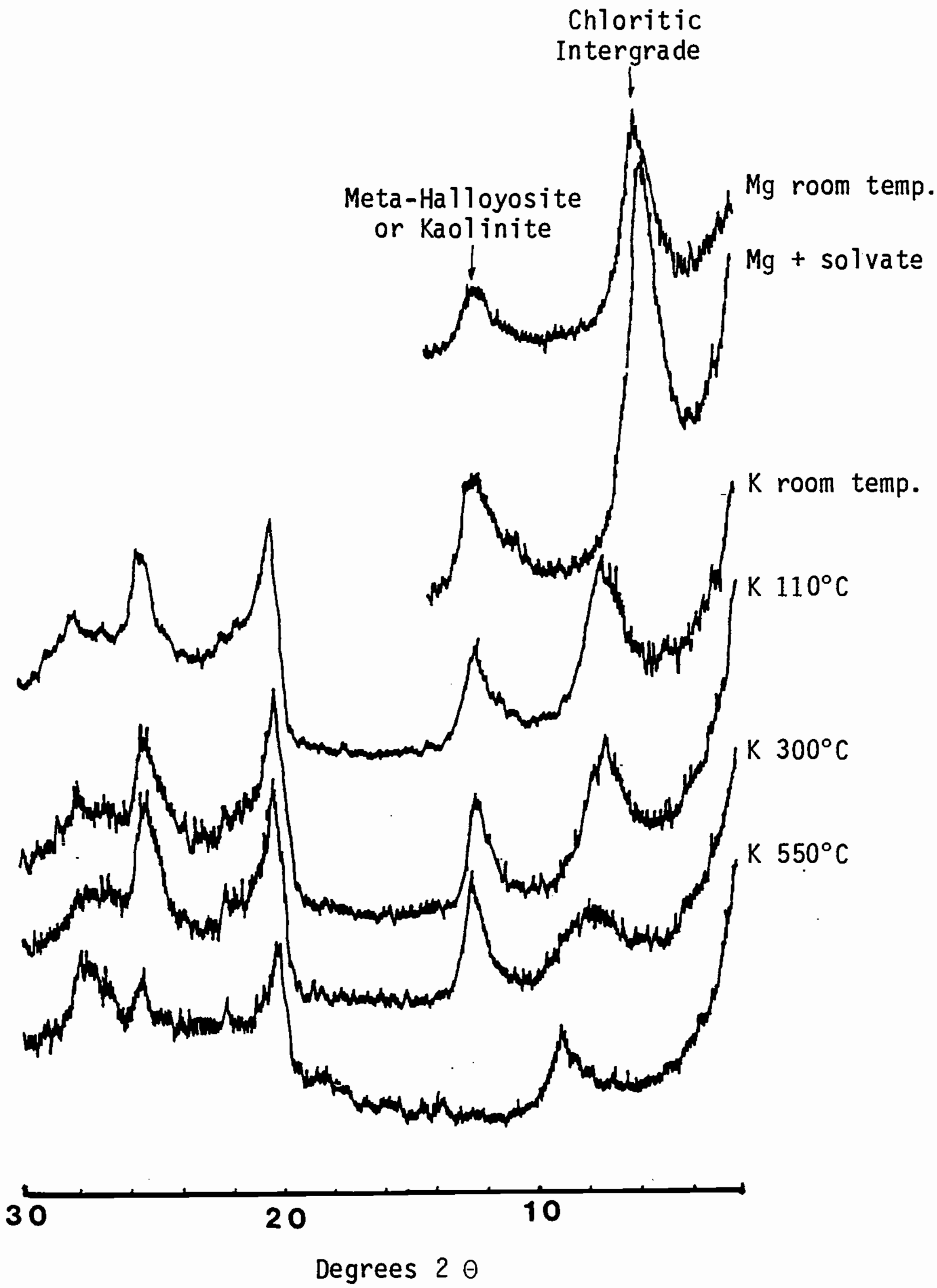

Figure 33. X-ray diffraction of brown MH silt from toe sample site E.

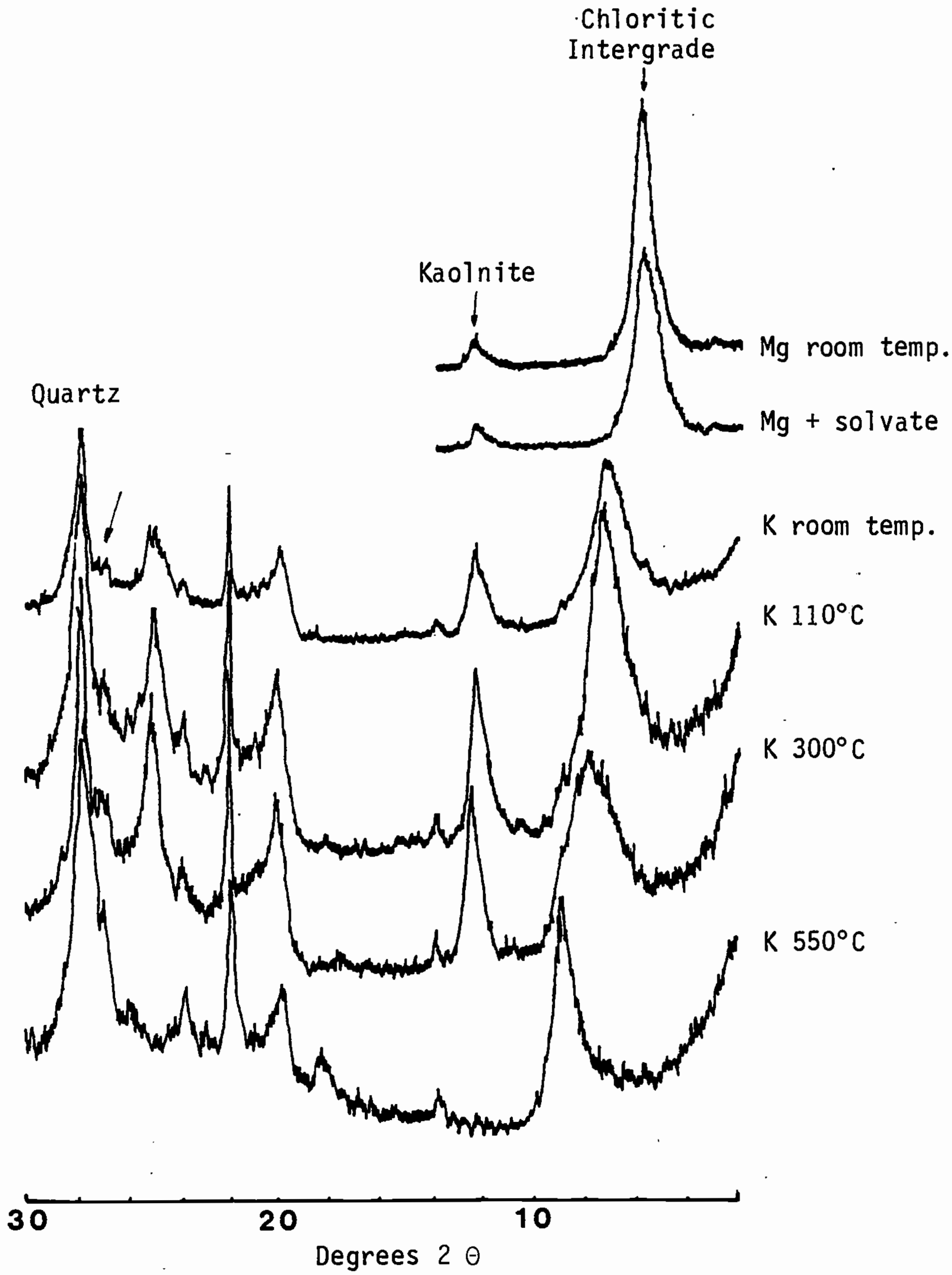

Figure 34. X-ray diffraction of green sandy (debris flow) colluvium.

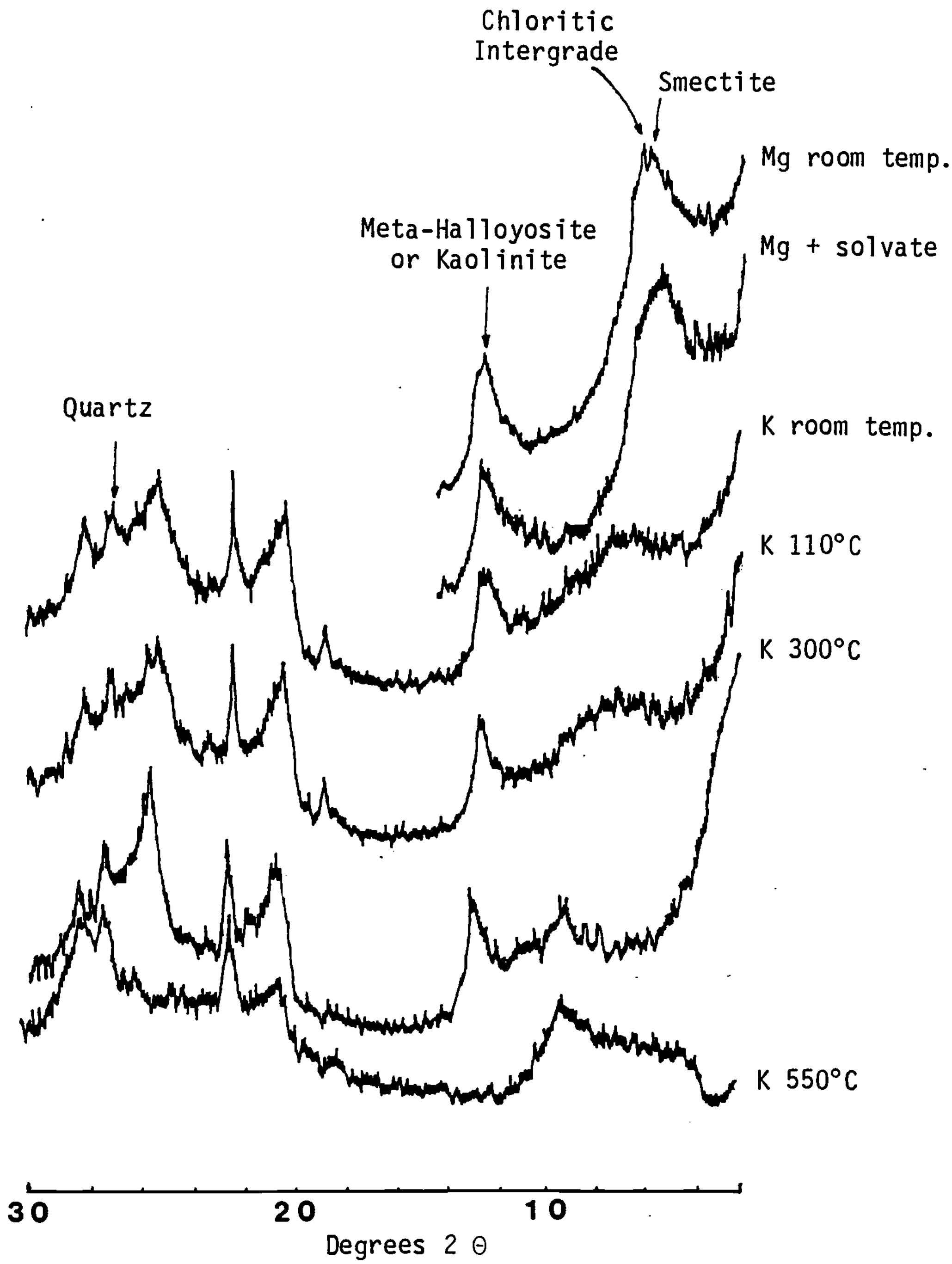

Figure 35. X-ray diffraction of sample no. 25, borehole No. 1, gray brown silt, 16 ft. depth.