Soil Engineering Properties and Vegetative Characteristics for Headwall Slope Stability Analysis in the Oregon Coast Range

by

Mark Bransom

A THESIS

submitted to

Oregon State University

in partial fulfillment of
the requirements for the
degree of

Master of Science

Completed November 29, 1990

Commencement June 1991

AN ABSTRACT OF THE THESIS OF

Mark Bransom for the degree of Master of Science in Forest Engineering (Hydrology) presented on November 29, 1990

Title: Soil Engineering Properties and Vegetative Characteristics for Headwall Slope Stability Analysis in the Oregon Coast Range.

Abstract Approved: ______________________
Henry A. Froehlich

Six headwalls in the central Oregon coast range were selected for study of soil engineering properties and vegetative characteristics important for analysis of slope stability. The headwalls were considered representative of those which would be candidates for timber leave areas due to geomorphic and topographic features, including steepness of slope and presence of a well defined constriction point on the downslope end. Of the six, only one headwall had evidence of recent debris-avalanche activity.

No standing overstory species were found on any of the headwall blocks, the area of the headwall thought most susceptible to failure. Douglas-fir (*Pseudotsuga menziesii*), maple (*Acer sp.*), and red alder (*Alnus rubra*) were common on the headwall perimeters. Salmonberry (*Rubus spectabilis*) and swordfern (*Polystichum munitum*) dominate the understory plant community on the headwalls.

Subsoils from two profiles on each headwall were sampled for index and classification properties. The soils are typically shallow, averaging under one meter in depth. They are silt-sand-gravel mixtures, with non-plastic fines. The soils all have exceptionally low density, and correspondingly high void ratio.

Consolidated-undrained triaxial tests with pore pressure measurements were conducted using relatively undisturbed samples to determine the effective soil strength parameters, angle of internal friction, ϕ', and cohesion, c'. Low consolidation pressures were chosen to model field overburden conditions. The arithmetic mean ϕ' was 31.1°. All the soils are considered to be slightly- to non-cohesive. There are no apparent correlations between index or classification properties and strength parameters. Variability of index properties and strength parameters appears to be as great within a particular headwall as between the six study sites.

Variability in the results of the strength tests are thought to reflect natural variability resulting from colluvial soil formation processes, as well as difficulties inherent in collecting and testing relatively undisturbed soil samples. Further investigations of strength parameters designed to include root contribution to strength are needed in order to more fully define stability characteristics of headwalls.

APPROVED:

____

Professor of Forest Engineering in Charge of Major

____

Head of Department of Forest Engineering

____

Dean of Graduate School

Date thesis is presented: November 29, 1990

Prepared by researcher for: Mark Bransom

Acknowledgements

The author wishes to acknowledge the guidance provided by the major advisor, Dr. Henry Froehlich, an infinite source of wisdom and patience. I am grateful for the opportunities provided me. Gratitude is extended to Dr. Marvin Pyles for critically reviewing the work, and teaching that skill to the student. Thanks are extended to Dr. Peter Clark for acting as minor professor and advisor. The supplemental support of the Mary J.L. McDonald Fellowship Fund, and the Dorothy D. Hoener Memorial Fund is greatly appreciated.

Secondly, words of thanks go to my colleagues who eased the burden by sharing their knowledge and skills, and for engaging me in many an insightful discussion which served to remind us that we are members of a global community.

Lastly, I wish to thank my wife, Kay, a true friend, also a colleague, and a steadfast force, for helping me to pursue my vision.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Soil Engineering Properties and Vegetative Characteristics for Headwall Slope Stability Analysis in the Oregon Coast Range

Introduction

Mass wasting processes are a significant geomorphic agent shaping the landscape in the Oregon Coast Range. The coastal mountains are characterized by high relief, with very steep slopes and narrow ridges and valleys. In these areas soil mass movement is considered to be the most important erosional process (Brown, 1973). Naturally occurring mass movements, including rock slides, debris avalanches and debris flows, slumps, and soil creep, are common in undisturbed areas of the coastal mountains. In addition to natural events, timber harvesting activities may accelerate mass erosion rates above natural levels. Swanston (1974) has hypothesized that accelerated rates of erosion in harvest units may result from loss of root reinforcement of soil over time, and removal of surface vegetation cover which can increase overland flow.

A number of landslide inventories in the central Oregon Coast Range indicate greater rates of mass erosion within clearcut units than in forested areas (Ketcheson and Froehlich 1978, Swanson et al. 1977). Different methods of determining these rates have produced a wide range of reported results.

In addition to timber harvesting, road construction activities have been recognized as a principal source of accelerated erosion in steep forest lands (Schroeder and Brown 1984, Burroughs et al. 1976). Slope failures associated with road building often result from slope loading, over-steepened bank cuts, and inadequate provision for slope and road drainage (Swanston, 1974).

Landslides are of concern because of the impact to natural slopes and transportation systems, and the potential for damage to downslope values. Landslides can remove areas from timber production through removal of topsoil and can disturb stream systems and riparian zones. Soil and debris entering a stream channel have the potential to affect fish habitat and migration routes through increased stream sediment and debris load, and can damage structures such as stream crossing culverts and bridges.

Landslide mitigation activities traditionally focus on recognition and avoidance of high risk, slide-prone sites. In addition to relocating proposed transportation routes, leaving standing vegetation on unstable portions of harvest units is now in practice. These so-called "leave areas" are thought to reduce the risk of failure by some combination of root reinforcement, anchoring, and buttressing of soil. This technique, in which the vegetation on slide-prone sites is undisturbed during harvest and site preparation, can be costly in terms of lost timber revenue.

Leave areas can also result in added costs through inefficient harvest operations if harvest rigging must be reset to avoid these areas. Lastly, the proper placement of leave areas relies on correct identification of the highest risk sites.

One aspect of recent slope stability research in forested areas has focused on the benefits of leave areas in headwalls. Investigations to date suggest that headwall leave area failure rates are similar to those in harvested headwalls, as well as being similar to failure rates in undisturbed forest (Froehlich, 1989). Because of the methods by which the various studies have been carried out, it is difficult to determine whether this finding contradicts the results of landslide inventories which suggest greater mass erosion rates in harvested sites. However, it is probable that a non-homogeneous sample biases a comparison of failure rates in headwall leave areas, harvested and forested headwalls. Because of the subjectivity of the risk rating methodology used in determining which headwalls were candidates for leave areas, a range, from low to very high risk sites with different degrees of downslope damage potential, are being compared. If the leave areas are stratified by a common risk rating method, the bias could be minimized and some measure of the success of leave areas gained. Due to the small population of leave areas, and the short time-frame over which they have been studied to date, it is difficult at this time to determine whether leave areas function according to design.

Alternatives to risk rating methodology for evaluating headwall stability include deterministic slope stability models, and combined deterministic-probabilistic models. Currently, use of mechanistic methods of identifying and characterizing high-risk sites are limited by a lack of comprehensive data on factors affecting slope stability in headwalls. In order to improve identification of high risk areas, site specific characteristics must be analyzed. This research will provide information appropriate for evaluating headwall slope stability. Vegetative, topographic, and geologic features can provide clues to potential stability problems. Geotechnical data, including soil strength parameters and groundwater conditions, typically serve as a basis for analyzing slope stability. Mechanistic models can then be employed to carry out these analyses, and a probability of failure or a factor-of-safety (FS) for a slope can be obtained.

It is the objective of this thesis to collect site information from a sample of steep headwalls in the central coastal mountains of Oregon, and provide soil engineering property data and vegetative and topographic characteristics which may be useful in evaluating slope stability. The variability of soil properties both between and within the study headwalls will be investigated.

Currently there exists a scarcity of such data from headwalls in the Oregon coast range. Therefore, it is the authors' intent that this investigation provide a foundation for more detailed study of headwalls, as is necessary for determination of landslide hazard and the success of mitigation measures.

Literature Review

Hillslope Morphology and Failure Theory

The stability of forested slopes is of concern to land use planners and natural resource managers because of potential damage to downslope values resulting from slope failures. The ability to identify and characterize landslide prone sites is increasingly important to maintaining a broad range of forest and riparian zone values. As a means to that end, it is important to recognize the forces acting on natural slopes, and how the forces affect the stability of the slope. In this way, slope response to changes in principal forces may be predictable. Changes in the principal forces may result from such things as a rise in the water table, removal of vegetation, or addition of side-cast to the slope from road construction.

The degree of instability and the type of failure likely to occur can often be associated with certain structural, geomorphic and topographic features. Uplift along the Oregon coast has created areas which are inherently unstable due to their geologic character. Among the structural features of concern for slope stability are fault zones, where accumulated stress is released and intrusive geologic materials may be found. These areas often contain rock that is fractured, crushed, or partially metamorphosed (Burroughs et al.,1976). These weak materials are often associated with zones of instability. In non-faulted areas, rock strength remains a structural consideration. The materials' competence, its natural resistance to deformation under compressional and tensile forces, is a measure of its strength. A third indicator of potential instability is the slope of bedding planes. The greatest degree of instability often occurs when bedding planes are approximately parallel to the slope of the soil surface (Burroughs et al. 1976, Glock 1979).

Hillslope morphology can also indicate areas of potential instability. Coast range hillslopes are frequently dissected by small unchanneled valleys. Hillslopes can be classified as planar, convergent, or divergent. Dietrich (1989) refers to these three elements as side slopes, hollows, and noses respectively. Steep hollows in the upper reaches of the unchanneled valleys are commonly known as headwalls, and are sometimes referred to as zero-order basins to suggest an undivided channel initiation reach. The bowl-shaped headwalls are thought to be sites of long-term accumulation of colluvial soil and rock material resulting from depositional processes (Dietrich and Dunne 1978, Dietrich et al. 1986). Headwalls are thought to be particularly susceptible to debris avalanche-debris flow type failures due to particular physical characteristics.

These may include shallow, colluvial soils, and concave outward bedrock which may promote convergent subsurface drainage and rapid groundwater rise in response to precipitation (Burroughs et al. 1976, Fredriksen and Harr 1979, Reneau and Dietrich 1987, Swanston et al. 1983, Swanson et al. 1977).

Where shallow soils occur on steep slopes, most mass-wasting is thought to occur as debris avalanche-debris flow landslides (Burroughs et al. 1976). Landslides of this form have been defined by Bailey (1971) as:

> "...the rapid downslope movement of incoherent soil, rock, and forest debris with varying water content. The detached soil mantle slides downslope above an impermeable boundary within the loose debris or at the unweathered bedrock surface and forms a disarranged deposit at the base. Downslope, a debris avalanche frequently becomes a debris flow because of substantial increases in water content.. ..They are caused most frequently when a sudden influx of water reduces the shear strength of earth material on a steep slope, and they typically accompany heavy rainfall."

Patric and Swanston (1968) and others suggest that rainfall is probably the main driving force determining when debris avalanches occur, but that topography, geology, and soil factors usually determine where debris avalanches occur.

Soil Strength and Effective Stress Theory

Yee (1975) suggests that the hydrologic behavior of earth materials and their shear strength are the most useful properties for analysis of many types of slope stability problems in the coastal Oregon mountains. Principle among those properties which can be measured, and provide an indication of a soils' hydrologic behavior is hydraulic conductivity, 'K'. Those properties providing an indication of strength include: particle size distribution, void ratio, 'e', angle of internal friction, 'ϕ', and cohesion, 'c', including root cohesion. Hydraulic properties and strength parameters are intimately related to one another, and to the stability of a sloping soil mass. The presence of water, and the hydraulic properties affecting the transmission of water through the soil have a direct affect on the materials' strength. A discussion of the affect of water on soil strength follows.

Implicit in the description of debris avalanche-debris flows given by Bailey above, is that such mass movements result from changes in the relationship of shear strength of the earth material and the shear stress on a failure surface within the profile. In cohesionless materials, slides occur by shear failure when accumulated stresses exceed frictional resistance to shear. The resistance to shear in such materials is produced by intergranular friction, which is usually represented by the angle of internal friction, ϕ. The angle of internal friction is largely a function of angularity, arrangement, size, and surface roughness of the soil grains, as well as grain size distribution (Terzaghi and Peck, 1967). In soils having appreciable fines, shear strength may result from cohesion as well as friction. In addition, an apparent cohesion is thought to be imparted to soils by roots through binding and anchoring (Bishop and Stevens 1964, Swanston 1970, Ziemer 1981).

The relationship between soil shear strength, and cohesion and angle of internal friction can be expressed by Coulomb's law (Holtz and Kovacs, 1981):

$$\tau = c + \sigma \tan \phi \qquad (1)$$

where τ is the shear strength, c is the cohesion, σ is the normal stress (perpendicular to the failure plane), and ϕ is the angle of internal friction. Expressed as tan ϕ, the angle of internal friction is analogous to the coefficient of friction (Yee and Harr, 1977).

Prior to the development of the concept of effective stress (Terzaghi, 1943), the affect of water on soil strength was not well understood. Terzaghi recognized that in saturated soils pore water bears a portion of the total normal stress acting on the soil mass. Because the water cannot support static shear stress, he termed the pore water pressure the neutral stress, u. The portion of the normal stress supported by the soil particles Terzaghi called the effective stress, σ'. The total normal stress, then, is the sum of the effective stress and the neutral stress:

$$\sigma = \sigma' + u \tag{2a}$$

rearranging,

$$\sigma' = \sigma - u \tag{2b}$$

Applying the effective stress principle to the Coulomb equation (Equation 1), the affect of the neutral stress in reducing τ becomes apparent:

$$\tau = c + (\sigma - u)\tan \phi \tag{3}$$

In terms of effective stresses, Equation 3 generally has the form:

$$\tau = c' + \sigma'\tan \phi' \tag{4}$$

The significance of the effective stress principle for geotechnical work cannot be over-emphasized.

Holtz and Kovacs (1981) stated:

> " We believe the effective stresses in a soil mass actually control or govern the engineering behavior of that mass. The response of a soil mass to changes in applied stresses depends almost exclusively on the effective stresses in that soil mass."

As the effective stress in a soil cannot be measured, but must be calculated, tests of effective strength parameters require that pore fluid pressures be monitored, as well as applied principal stress differences. Triaxial tests of soil shear strength are commonly conducted with such measurements. See Bishop and Henkel (1962) for a detailed treatment of triaxial soil strength testing theory and practice.

Once strength parameters of a material are known, a measure of the stability of a slope in that material can be gained from a limit equilibrium analysis. A factor-of-safety (FS) produced by a limit equilibrium analysis is the ratio of available resisting forces and the forces driving failure. A FS of 1 suggests the slope is in a state of limit equilibrium. As the FS increases above 1, there is a greater degree of certainty that the slope will be stable. An alternative definition of the factor-of-safety is that FS is the ratio of the strength of the material to the stresses on a failure plane driving failure.

Such an analysis is important in order to judge the relative stability of the slope, as well as provide a means of predicting slope response to changes in the principle forces acting on the slope. Currently insufficient data from headwall soils exists for such analyses to be made with any degree of certainty.

Previous Work

A number of previous studies have addressed the relationship between soil index properties and strength parameters, and mass-wasting processes on steep forested slopes (Yee 1975, Burroughs et al. 1976, Fredriksen and Harr 1979, Alto 1982, Schroeder and Alto 1983, Schroeder and Brown 1984). Based, in part, on such work, several landslide hazard rating and headwall stability models have been proposed (Burroughs 1984, Sidle 1987). Model parameters include: topographic characteristics of the headwall; soil strength parameters; an assessment of apparent soil reinforcement imparted by roots along potential failure planes; and groundwater conditions during a design storm. The Three-Dimensional Level I Stability Analysis (3DLISA) model (Burroughs, 1984) requires the number, species, and location of trees, and composition of understory vegetation on the headwall as a means of assessing root reinforcement.

In a slope stability analysis, 3DLISA uses Monte Carlo simulation of input parameters to produce a probability distribution of factors-of-safety. The probability of failure of the slope is the ratio of the number of factors-of-safety less than 1, to the total number calculated. Monte Carlo simulation is designed to account for uncertainty associated with the model parameters through use of a mean value and some measure of dispersion about that mean. Management level application of the model in headwalls is of limited value at this time, as no satisfactory estimate of these parameters currently exists.

In a literature review of Pacific Northwest slope stability research, no previous studies of soil index properties or strength parameters in headwalls was found in the published literature. However, several studies of index and engineering properties of Coast Range soils can serve as a basis for exploring questions of what one may expect to encounter in headwalls. A review of these investigations follows.

Of the two soil types tested by Yee (1975) both were extremely porous, very well aggregated and very well graded, highly permeable, sandy to gravelly, cohesionless soils. The soils tested were the sandstone derived Bohannon series, and the gabbro derived Klickitat series. Both had low unit weight, and high total porosity, which was attributed to the high degree of aggregation found to be common among the soils. While Yee (1975) found the soils to be well graded, he noted that the particle size distribution of the soils was dependent on the intensity of the dispersion technique used in making the analysis. Schroeder and Alto (1983), testing soils similar to those of Yee, noted that additional mechanical energy input during grain size analysis resulted in continued breakdown of the soil particles to yield finer distributions. Yee concludes from his observation that the field grain size distributions are probably coarser than reported. From the perspective of the soil engineering behavior, however, the finer distribution may be most critical. Assuming aggregate disintegration during wetting occurs in situ as it did in Yee's laboratory tests, the field grain size distribution could then be dependent on soil-water conditions.

Yee (1975) found unusually high values of the dry angle of internal friction, ϕ. Values of ϕ were determined to be approximately 40° for all soils. Terzaghi and Peck (1967) list representative values of ϕ for similar, loose sands to be in the range of 27° to 33°. Again, Yee (1975) considered stable aggregates the likely reason for the high values. It should be noted that the values given by Terzaghi and Peck (1967) are considered to represent a conservative range, for use in typical civil engineering works. Schroeder and Alto (1983) conducted consolidated-undrained triaxial tests with pore-pressure measurements and reported that soils tested from the Oregon and Washington coastal mountains had an average ϕ' of 37.8°. While also a conservative estimate, this value compares more closely with that of Yee. Schoenemann and Pyles (1990), using a statistical procedure for describing shear strength test results, reported a ϕ' of 39.9° for the Schroeder and Alto test data.

Most soils involved in debris avalanches are, or are assumed to be, cohesionless (Bishop and Stevens 1964, Harr and Yee 1975, Swanston 1970, Ketcheson 1978). Cohesion values reported by Schroeder and Alto (1983) ranged from non-cohesive to slightly cohesive with values of c' up to 6.87 kPa. Yee (1975) reported all soils tested were cohesionless. Apparent cohesion imparted to soils by roots through binding and anchoring is thought to be a significant component of soil shear strength (O'Laughlin 1968, Ziemer, 1981). These studies suggest, however, that root strength declines within four to five years of timber removal from a site. While the full implication of root contribution remains unclear, and is beyond the scope of this study, the contribution of roots to earth material strength points out the need to conduct strength testing of the combined soil-root system when appropriate, in order to more fully understand the role of root strength in slope stability.

Implications for Slope Stability Analysis

The results of some slope stability modelling procedures can be shown to be highly sensitive to changes in the input parameters, including soil strength parameters (Megahan and Gray, 1981). In order to more accurately assess the validity of stability analyses of a sloping soil body, the affects of soil sampling and testing method on the results of strength parameter tests must be considered.

Yee (1975) reported that there was a significant difference in values of dry and saturated angle of internal friction from triaxial tests. This was attributed, in part, to the breakdown of soil aggregates during wetting in the saturated tests. This suggests that the results of strength tests for some soils are dependent on the method of testing, and that disturbance of the sample can affect test results.

Burroughs (1984) discusses the difference in soil strength test results by laboratory and field methods for central Oregon coast range soils. A comparison of laboratory triaxial tests (Schroeder and Alto 1983), with field tests using a bore-hole direct shear device described by Handy and Fox (1967), showed the field measurements yielding higher values of effective angle of internal friction, ϕ', than the triaxial tests. Part of the difference in test results is explained simply by the different test methods, and part is caused by direct shear tests of unsaturated soils, which will give higher strength parameters (Burroughs, 1984). Referring to Equation 3, an increase in strength can be seen to result from a negative value of u, which is the condition in unsaturated soil. In addition to different testing methods, collecting soil samples for laboratory analysis invariably results in disturbance of the natural soil structure. Similarly, the difficulty in obtaining a satisfactory bore-hole in relatively cohesionless soils introduces a testing complication. The affect of these factors on the results of strength tests is unknown.

Analysis of a large enough data base may reveal that some measure of soil strength parameters could be determined by correlation with soil index properties. It may also be possible to correlate strength parameters obtained by simpler, yet less reliable, techniques, with those obtained from standardized procedures. Tests of soil index properties could provide a check on strength tests, or provide a means of estimating strength parameters without encountering the difficulties of strength testing. Based on a small sampling, Schroeder and Alto (1983) report that laboratory data from soils of the Oregon and Washington coast ranges indicate that effective strength parameters could not be correlated with index properties of those soils, in part, because of the relatively large number of variables which influence soil strength.

The implications of the previous discussions for slope stability analysis are that hydrologic behavior and the relationship of shear strength and shear stresses in the soil are most likely to determine the stability of a sloping soil mass (Yee, 1975). Information regarding these characteristics, and the variability expected to be found, are fundamental for the analysis of slope stability in any meaningful fashion. In addition, various sampling techniques and testing methods can lead to disparate results of strength parameter tests. Further, when strength parameters obtained by dissimilar methods are used to determine a FS, or failure probability, very different pictures of the stability of the material may result. Consequently, the means by which strength parameters were obtained must be considered when interpreting results of stability modelling procedures, and caution used in assigning a single value to a model parameter.

Study Areas

The headwalls selected for study lie within the Mapleton District of the Siuslaw National Forest, in the central Oregon coastal mountain range. The Mapleton District extends from Heceta Head in the north to the Umpqua River in the south, and includes the west side of the coast range. Six headwalls were selected for study. Three sites, designated HW1, HW2, and HW3, are located in the northern portion of the district, at headwaters of tributary streams of the Siuslaw River, near the town of Mapleton. An additional three headwall study sites, designated HW4, HW5, and HW6, are located between the Smith River and the Umpqua River, in an area known as the Smith-Umpqua Block in the southern portion of the district. The study site locations are shown in Figure 1.

The central coastal mountains of Oregon are geologically young, and characterized by high relief (150-600 meters). Steep slopes (40-100%) and narrow valleys are common throughout the range. The Oregon coast is an active margin where the Juan de Fuca plate is being subducted under North America. This convergence, which is thought to have begun 30 million years ago, has caused uplift of the Oregon Coast Range (Dietrich, 1989).

The mountains are composed primarily of Cenozoic marine sedimentary rocks. The Mapleton District is underlain by the Tyee (Flournoy) Formation (Baldwin, 1976). The Tyee is composed of massive, rhythmically bedded micaceous and arkosic sandstone and sandy siltstone. Soils overlying bedrock are colluvial deposits or residual soils formed from weathering of the near surface sedimentary rocks. Many of the soils on the steepest slopes are formed from colluvium and frequently are thin and stony, exhibiting a relatively homogeneous profile due to transportation and thorough mixing (Pierson, 1977). Since uplift of the coastal range began, the drainage network has been actively downcutting, draining the region to the Pacific Ocean to the west, and to the Willamette River valley to the east. Large rivers such as the Siuslaw, Smith, and Umpqua Rivers have cut through the uplifting rock, giving way to new valley and ridge terrain features. Sea level fluctuations, together with tilting and subsidence of the structural units, has led to a complex history of base level changes which may have propagated upstream into the coastal mountains (Dietrich, 1989).

The region's climate is dominated by marine air masses from the Pacific Ocean. A Mediterranean type climate prevails, with a dry season extending from May through September, and a wet season from October through April. Average annual precipitation ranges from 150 to 300 centimeters, mostly in the form of rain (NOAA Climatic Data for Oregon). Snow deposits commonly occur, but persist only at higher

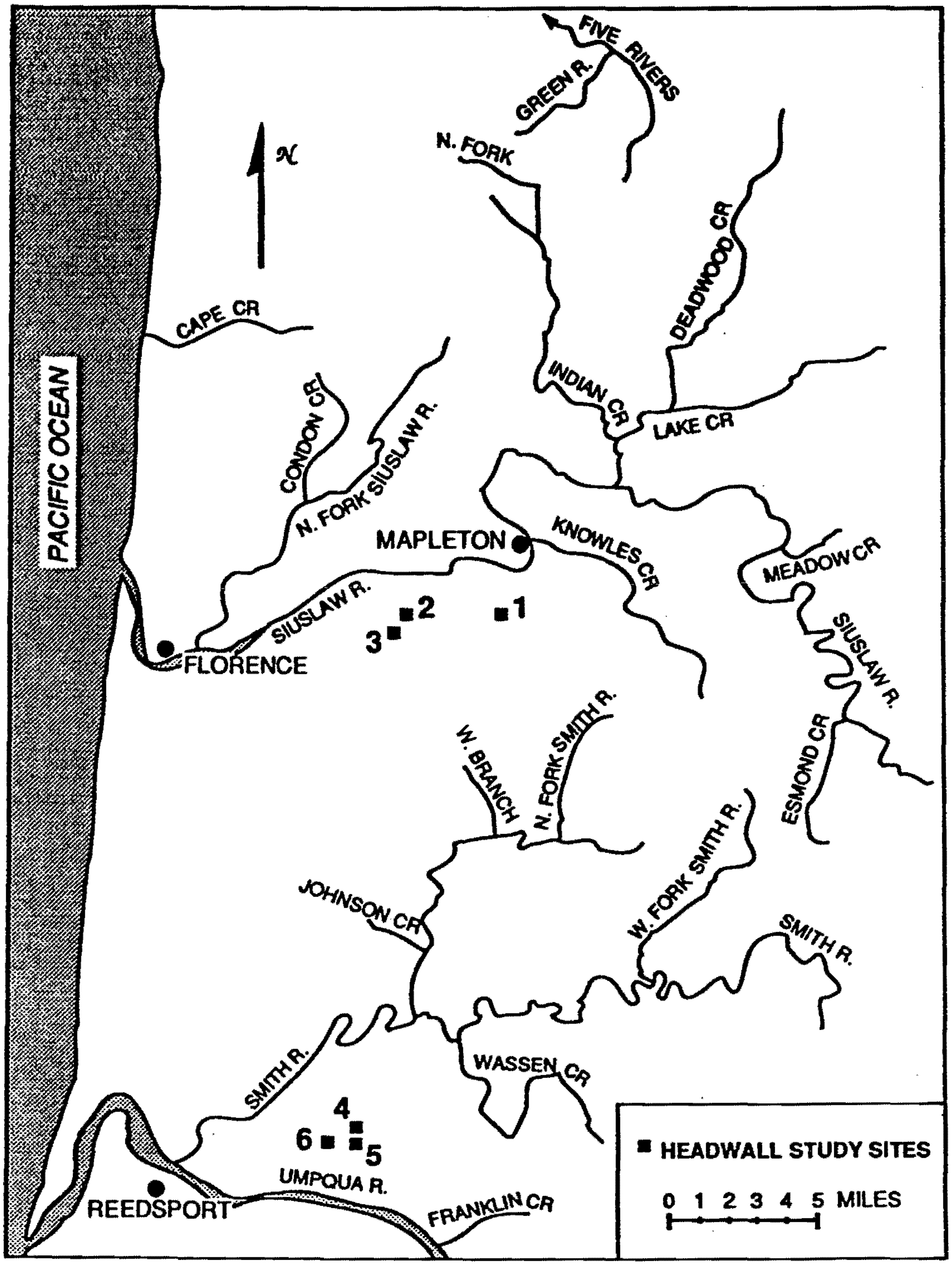

Figure 1. Headwall study site location map.

elevations. Prolonged periods of freezing temperatures are rare.

The region supports extensive forests consisting primarily of Douglas-fir (Pseudotsuga menziesii), western hemlock (Tsuga heterophylla), and western redcedar (Thuja plicata) as the principal overstory species (Ketcheson and Froehlich, 1978). Red alder (Alnus rubra), and big leaf maple (Acer macrophyllum) are also common. Numerous brush, fern, and forb species occupy the understory. In addition to a diverse land based ecosystem, many of the streams and rivers support native and anadromous fisheries.

Research Methods

Sampling Methods

Six forested headwalls were selected for study of soil index and engineering properties, geomorphology, and vegetative characteristics. Selection of the study sites was based on several criteria:

1) the headwalls were in forested areas;
2) the sites were very steep (≥80%), and had a definite constriction on the downslope end;
3) the headwalls were not disturbed by upslope roads;
4) the headwalls were within 10 kilometers of a recording rain gage.

Criteria 1 and 2 above are based on the premise that headwalls of this type would be eligible for headwall leave areas if included in a timber sale on federal lands. Timber harvest guidelines for many slide-prone sites stipulate that standing timber and understory vegetation must be undisturbed, if headwalls having a particularly high risk of failure are identified. The remaining criteria ensure minimal human induced disturbance (3), and an opportunity to estimate precipitation on the study sites if desired (4). Criteria 4 was established concurrently with the beginning of precipitation measurements with the rain gage network; subsequent analysis of precipitation data from gages in the study area suggest the 10 kilometer distance may be too great, given the apparent high variability of precipitation over short distances in the coast range.

A detailed record was made of the site characteristics and the soil sampling. Headwall site and soil profile descriptions are given in Appendix A. Among the information recorded at each headwall was dimensional and topographic features of the headwall, vegetation species distribution and composition, and soil characteristics.

The headwalls, as previously defined, are concave outward depressions, and are bounded on the sides by topographic divides. The headwall "block" is the region of the headwall thought to be most susceptible to failure. As used herein, the headwall block is defined as the soil and vegetation extending from the constriction or "critical point" at the lower end, to an upper boundary at which a slope-break occurred, and laterally to the base of the sideslope at the topographic divide on either side of the headwall, and bounded by bedrock at the base. Appendix B shows a typical headwall contour map with the headwall block and other features delineated. The critical point, as used in the slope stability model 3DLISA, is defined as the most likely failure initiation point, occurring where the headwall block narrows and subsurface flow concentration is likely to be greatest. Below the critical point the valley is frequently channelized. There is some degree of uncertainty as to the validity of the arguments used in defining such the critical point in a headwall. One, several, or no such points may be recognized in any particular headwall.

After the dimensions of the headwall block were determined, a vegetation map was made. Distinct regions of understory vegetation within the headwall were delineated. Regions were drawn on the basis of the dominant species present. No attempt was made to determine species mix, beyond dominant species, in the various delineations. Location of overstory species on, or next to, the headwall block was recorded. A headwall map with regions of distinct vegetation types shown, is given in Appendix B.

After the vegetation mapping was completed, brush over several square meters on the headwall block was removed with a chainsaw, in order to facilitate soil sampling and movement around the site. Two soil trenches were then dug to bedrock on each headwall, using hand tools. The trench near the top of the headwall block was designated Soil Profile 1; the trench near the critical point was designated Soil Profile 2. The two part site identification code (HW1-1 for example) used throughout this paper, reflects the headwall designation and the location of the soil profile on that headwall.

Based on observations of headwalls which have had slides, the bedrock-subsoil interface appears to be a common lower boundary of the slide block. In the absence of definitive information regarding the location of slide initiation, we have chosen to investigate properties of the subsoils. Therefore, samples were collected from near the base of the soil profiles, with the exception of one headwall. Both soil trenches at Site HW2 revealed two distinct horizons making up the soil profile. The surface soil, extending down to 95 cm was considered an "active" colluvial layer. This horizon was dark and appeared thoroughly mixed, having all the characteristics of a true colluvial soil. The subsoil, 95-152 cm, was a reddish layer which appeared to be stable and weathering in place. This interpretation was based on observation of soil pedon structure common to residual soils. The surface colluvial horizon was sampled for all tests.

Cross-slope length of the exposed soil faces averaged one meter. Soil pedestals were excavated near the base of the soil profile in order to collect undisturbed samples. Thin walled steel tubes (Shelby tubes) 30 cm in length and 7.3 cm inside diameter were used. In order to minimize friction between the soil and tube, the tubes were aligned vertically on a pedestal and slowly pushed into the soil by hand, as material around the outside of the tube was trimmed away. When roots or cobble impeded the tube, an-

other pedestal was used and the process repeated. A minimum of two Shelby tube samples were collected at each soil trench. Tubes were sealed with plastic end-caps and wrapped with duct tape, then placed in padded boxes for transport to the laboratory. Representative disturbed samples were collected for index property and classification tests. Soils were bagged in large ziplock storage bags and labelled with identification markers. Samples were taken from the same depths as the undisturbed samples. An average of approximately 18 kilograms of material was collected from each profile. All samples were stored in a refrigerator at approximately
15.5° C until testing. Humidity was maintained by placing open pans of water in the refrigerator. Samples were stored for one to 14 months. Sampling of the headwalls began in early June, 1989 and was completed by July 1, 1989.

Testing Methods

Field and laboratory testing was undertaken to determine the index properties, soil classification, and effective strength parameters of the soils. Soil index properties determined were:

1) natural water content, w_n;
2) dry soil density, ρ_d;
3) specific gravity of soil solids, G_s;
4) Atterberg limits and indices:
   a) liquid limit
   b) plastic limit
   c) plasticity index
5) grain size distribution.

Procedures followed those described by Bowles (1978), with modifications of some tests. The soils were classified according to the Unified Soil Classification System (USCS) (Holtz and Kovacs, 1981). Laboratory tests were begun in July, 1989 and completed in August, 1990.

In situ Soil Density

Soil density determinations were made on site using a Cambell Pacific Nuclear Corporation Model MC-1 Portaprobe Nuclear Density/Moisture Gauge. Measurements were made at selected depths by placing the gauge on soil benches cut from the profiles. Soil natural water content samples were collected from the appropriate depths. Water content, w_n, was necessary for determination of dry soil density, ρ_d. The probe was not used to determine soil water content *in situ* due to water content measurements made with the gauge in a previous investigation which were not in agreement with results of standard laboratory procedures.

Specific Gravity of Soil Solids

The specific gravity of a substance, G_s is defined as the ratio of the unit weight of the substance to the unit weight of water at 4°C. Initial tests of specific gravity revealed substantial variability in results as a function of the amount of organic matter present. A modification of the Bowles (1978) procedure was undertaken to remove organics from the soil. Approximately 75 grams of oven dry soil was reacted with excess 5% hydrogen peroxide solution (H_2O_2) for 24 hours. The amount of H_2O_2 required to completely oxidize the organics varied, but averaged one liter. A second treatment with H_2O_2 was done when the initial reaction appeared to have ceased. After a complete reaction, the soil was covered with de-aired water and the mixture boiled for 10 minutes. After boiling, the mixture was transferred to a pycnometer and the test completed as per Bowles. One soil sample from each headwall was tested.

Atterberg Limits

One Atterberg limit, and one Atterberg index are required for classification of soils in the USCS. These are the liquid limit (LL), and the plasticity index (PI). The liquid limit is the water content at which the soil undergoes a transition from a plastic solid to a viscous liquid. The plasticity index is defined as the difference between the liquid and plastic limits. The plastic limit (PL) is the soil water content which separates a solid from a plastic solid state. The plasticity index, then, is the range of water contents over which the soil behaves as a plastic solid. The liquid limit of each sample was determined by a three point test. The plastic limit of each sample was determined by averaging results of two plastic limit tests.

A modification of the Bowles (1978) procedure was also employed in the Atterberg Limits tests. Soil samples used in determination of Atterberg Limits were initially at natural water content. The procedure of Bowles calls for the use of air-dried samples. Concern for irreversible changes in particle hydration after drying was the basis for the procedural modification. Studies suggest that Atterberg tests using moist soil, versus those using air dry soil, can yield different results (Holtz and Kovacs, 1981).

Grain Size Distribution

Mechanical grain size analysis was undertaken to determine the particle size distribution curve for each soil. Oven dry samples were washed on a U.S. Standard #200 sieve (75 μm mesh). Samples were then redried and sieved through a seven-sieve stack on a mechanical shaker. The sieve stack included U.S. Standard sieve numbers 4 (4.75 mm), 10 (2.00 mm), 20 (850 μm), 40 (425 μm), 60 (250 μm), 100 (150 μm), and 200 (75 μm).

No particle settling (hydrometer) analysis was undertaken to determine distribution of the material less than 75 μm. The rational for this is that the behavior of the fines, and not the distribution of sizes, is of primary importance to analysis of soil engineering behavior and slope stability. The degree of plasticity, and the mineralogy of the clay fraction are measures of the type of behavior expected of the fines. Therefore, estimates of these properties were used to characterize the fines, rather than simple size distribution.

Effective Strength Parameters

Multi-stage consolidated-undrained triaxial tests, with pore pressure measurements, were conducted to determine effective strength parameters ϕ' and c'. Figure 2 shows a schematic diagram of the testing apparatus.

Samples were extruded from the Shelby tubes into a vacuum operated triaxial membrane jacket containing a rubber membrane. A manually operated hydraulic jack was used to remove the soil from the tube. The sample ends were trimmed perpendicular to the long axis and water-saturated circular porous stones with filter paper were placed at each end of the soil sample. The sample was placed in the triaxial cell, and mounted in a loading frame. The soil was then flooded with de-aired water to displace air; a typical flooding stage lasted 30 minutes. After flooding, the sample was saturated under back-pressure. Skempton's pore pressure parameter 'B' (Skempton, 1954) was determined periodically. Back-pressure was increased in 35 kPa increments. A 'B' parameter value of 0.9877 or greater for sandy soils at most densities is considered to represent 100% saturation (Holtz and Kovacs, 1981). When 'B' attained this value or higher, back-pressure saturation was ended; this phase typically lasted 12 hours. Saturation was normally achieved at a back-pressure of 275 kPa. Due to a testing system bias, there was a difference between cell pressure applied to the sample and the back-pressure during the saturation phase which varied as a function of the back-pressure. In effect, the sample was experiencing some consolidation.

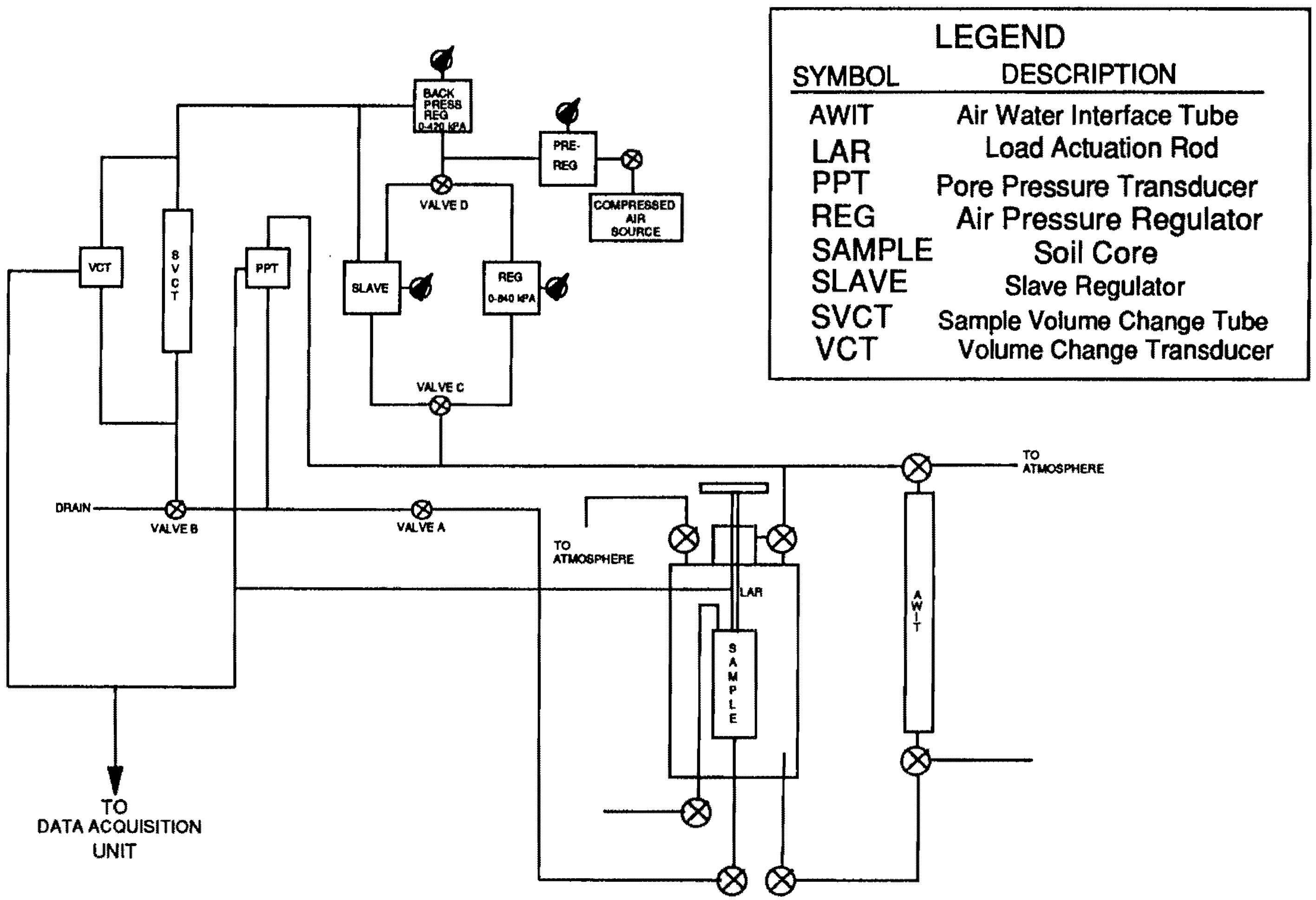

Figure 2. Schematic diagram of the triaxial testing apparatus.

The consolidation pressure at the end of back-pressure saturation was typically between 2.75 kPa and 3.50 kPa.

Following saturation, multi-stage consolidation and shearing of the sample was carried out. Due to the low *in situ* vertical confining stresses the soils experienced, the consolidation pressures selected were similarly low. Using the average soil depth and unit weight of subsoil from the study sites, the average total overburden stress was determined to be 12.8 kPa under the field conditions at sampling, neglecting the weight of vegetation. This value increases to 15.5 kPa overburden stress assuming saturated conditions. During the first consolidation phase, 7.0 kPa consolidation pressure was applied. After volume change readings stabilized consolidation was considered complete. Consolidation phases typically ran from 3 to 12 hours. The sample was then sheared using a strain controlled procedure. Strain rate was 0.05 mm/min.

Two additional consolidation and shear phases provided a three point effective strength envelope. The second and third stage consolidation pressures were 14 kPa and 21 kPa respectively. Real-time stress path and stress-strain plots were used to determine when to end a shear phase. The decision to stop loading the sample during stage 1 was based on observation of the stress path reaching a terminal point. Shear stages 2 and 3 were also ended when the stress path reached a terminal point; in addition, the information serving as basis for ending shear during stages 2 and 3 included observation of a slope break in the stress-strain curve, suggesting it was approaching a peak. Shear stages were typically 75 to 90 minutes in length.

Results

Site and soil descriptions are given in Appendix A. The vegetative features of the headwalls, and the results of the soil testing are reported below. A representative headwall topographic map with regions of distinct vegetation types shown is given in Appendix B.

Vegetative Characteristics

Of the six headwalls studied, none had any live, standing overstory species on the headwall block. One site, HW3, had one western redcedar (*Thuja plicata*) seedling on the headwall block. Several sites had vine maple (*Acer circinatum*) clumps of 5-13 cm diameter stems on the sideslopes of the headwall. Bigleaf maple (*Acer macrophyllum*) and red alder (*Alnus rubra*) were the most common overstory species, typically occurring at the topographic boundaries of the headwall. Douglas-fir (*Pseudotsuga menziesii*) was the next most common tree, and also occurred on the topographic boundaries of the headwall.

Two headwalls, HW2 and HW3, had three and two windthrown 60 cm diameter Douglas-fir trees on the block respectively, while HW4 and HW6, had one windthrown Douglas-fir each. The trees had not been rooted in the headwall block, but had fallen into it.

Colluvial material was observed to have accumulated behind those trees which were at an angle with respect to the long axis of the headwall. Buried wood was encountered during soil density measurements at HW1-1. Large pieces of wood and bark were found between 5 and 41 cm below the soil surface. Charcoal was also common throughout most of the soil profiles.

The dominant understory species found on the six headwall blocks were sword fern (Polystichum munitum), and salmonberry (Rubus spectabilis). The next most common species included hazelnut (Corylus cornuta), rhododendron (Rhododendron macrophyllum), huckleberry (Vaccinium ovatum and V. parvifolium), and thimbleberry (Rubus parviflorus). Other minor constituents included salal (Gaultheria shallon), ocean spray (Holodiscus discolor), red elderberry (Sambucus callicarpa), and stink currant (Ribes bracteosum). Various forb species formed nearly continuous mats over the soil surface in each headwall.

Understory species rooting depths were typically limited to the top 45 cm of the soil profiles. Large overstory species roots growing into the headwall from the sideslopes were observed in many of the soil profiles.

Soil Index Properties

A summary of index properties and soil classifications are shown in Table 1. The soils are typically non-plastic, silt-sand-gravel mixtures. Additional soil properties, including those determined from the index property data, are shown in Table 2.

Table 1. Soil index properties and USCS classification.

Site	w_n[a] (%)	G_s[b] -	ρ_d[c] (Mg/m^3)	LL[d] (%)	PL[e] (%)	PI[f] (%)	USCS Group
HW1-1	25.1	-	1.00	41.9	35.2	6.7	SM
HW1-2	31.6	2.72	1.10	49.9	40.4	9.5	SP-SM
HW2-1	32.1	2.66	1.14	49.6	44.4	5.2	SP-SM
HW2-2	58.0	-	0.92	51.1	47.0	4.1	SM
HW3-1	28.3	-	0.81	48.2	38.0	10.2	SM
HW3-2	44.6	2.69	0.79	53.8	43.6	10.2	SM
HW4-1	40.9	-	0.86	44.0	30.6	13.4	SM
HW4-2	30.7	2.70	1.11	37.2	28.5	8.7	SM
HW5-1	37.2	2.70	0.98	46.6	36.9	9.7	SM
HW5-2	36.1	-	1.00	47.2	36.3	10.9	GM
HW6-1	30.3	-	1.12	38.6	29.0	9.6	SM
HW6-2	33.3	2.71	1.10	44.7	32.2	12.5	SM

(a): natural water content
(b): specific gravity of soil solids
(c): dry density
(d): liquid limit
(e): plastic limit
(f): plasticity index

Table 2. Soil depth, unit weight, and void ratio for the study site soils.

Site	Total Soil Depth (cm)	Unit Weight, γ_d (kN/m^3)	Void Ratio e
HW1-1	140	9.81	1.72
HW1-2	67	10.79	1.47
HW2-1	152(a)	11.18	1.33
HW2-2	152(a)	9.03	1.89
HW3-1	58	7.95	2.32
HW3-2	46	7.75	2.41
HW4-1	101	8.44	2.14
HW4-2	94	10.89	1.43
HW5-1	64	9.61	1.76
HW5-2	98	9.81	1.70
HW6-1	116	10.99	1.42
HW6-2	79	10.79	1.46

(a): "Active" colluvial layer 0-95 cm, above stable soil layer 95-152 cm, sampled for all tests.

Effective Strength Parameters

Figure 3 shows a typical stress path plot obtained from the triaxial tests. The (p', q') coordinates of the stress paths can be defined in terms of principal effective stresses as follows:

$$p' = \tfrac{1}{2}(\sigma_1' + \sigma_3') \tag{5a}$$

and

$$q' = \tfrac{1}{2}(\sigma_1' - \sigma_3') \tag{5b}$$

where

σ_1' = major principal stress;
σ_3' = minor principal stress.

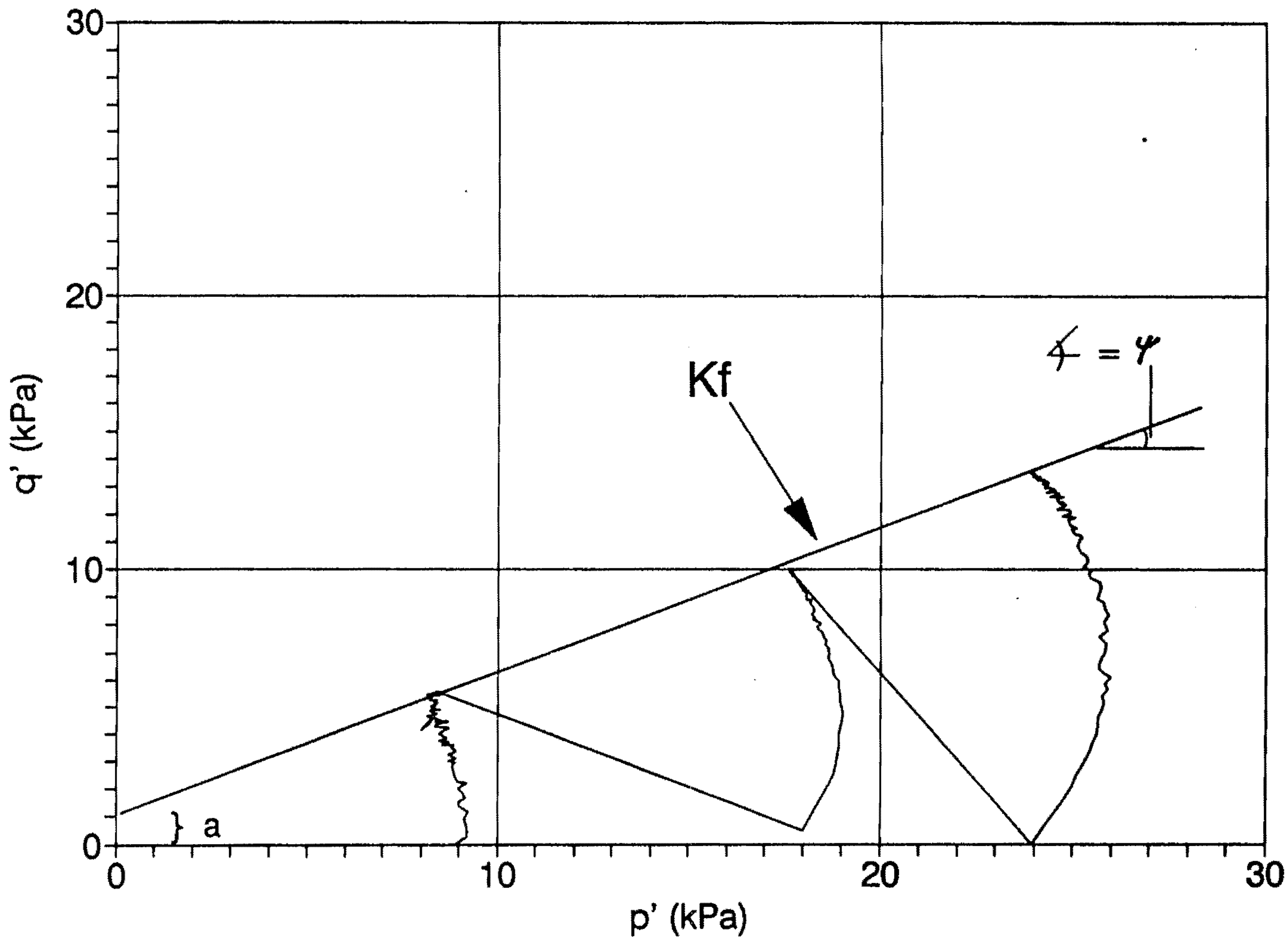

Figure 3. Typical stress path plot from triaxial tests of soil strength.

A stress path, then, represents successive states of stress as the sample was loaded during a shear phase. The failure line, K_f, is inclined at an angle, ψ', with respect to the horizontal. The angle of internal friction, ϕ' is related to ψ' by (Schoenemann and Pyles, 1990):

$$\sin \phi' = \tan \psi' \tag{6}$$

The q' intercept value, a', is related to cohesion, c', by:

$$c' = \frac{a'}{\cos \phi'} \tag{7}$$

Effective strength parameters, ϕ' and c', for the soils tested are summarized in Table 3. No results from HW5-2 are reported because all samples were too coarse to obtain strength parameters independent of particle size using these procedures.

In order to characterize the results of the strength tests, the statistical procedure described by Schoenemann and Pyles (1990) was used. All the samples were treated as one group for the analysis. Regression of q' on p' produced a K_f line and ϕ' for the group. One (p', q') point representing failure from the end of each stress path was then used to construct a regression in (σ', τ) coordinates.

Table 3. Angle of internal friction, ϕ', and cohesion, c', for the soils tested.

Site	ϕ' (degrees)	c' (kPa)
HW1-1	32.7	0.24
HW1-2	30.6	0.0
HW2-1	31.8	2.7
HW2-2	29.9	1.0
HW3-1	32.7	0.59
HW3-2	28.5	0.34
HW4-1	34.0	0.12
HW4-2	30.6	1.7
HW5-1	28.5	0.0
HW5-2	-	-
HW6-1	34.1	0.0
HW6-2	28.5	0.68

Normal stress and shear stress corresponding to each (p', q') coordinate were determined using the following equations (Schoenemann and Pyles, 1990):

$$\sigma' = p' - (q' \sin \phi') \tag{8}$$

and

$$\tau = q' \cos \phi' \tag{9}$$

Simple linear regression of τ on σ' was then carried out in order to define a representative Mohr failure envelope for the data set. Results of the regression procedure are shown in Figure 4.

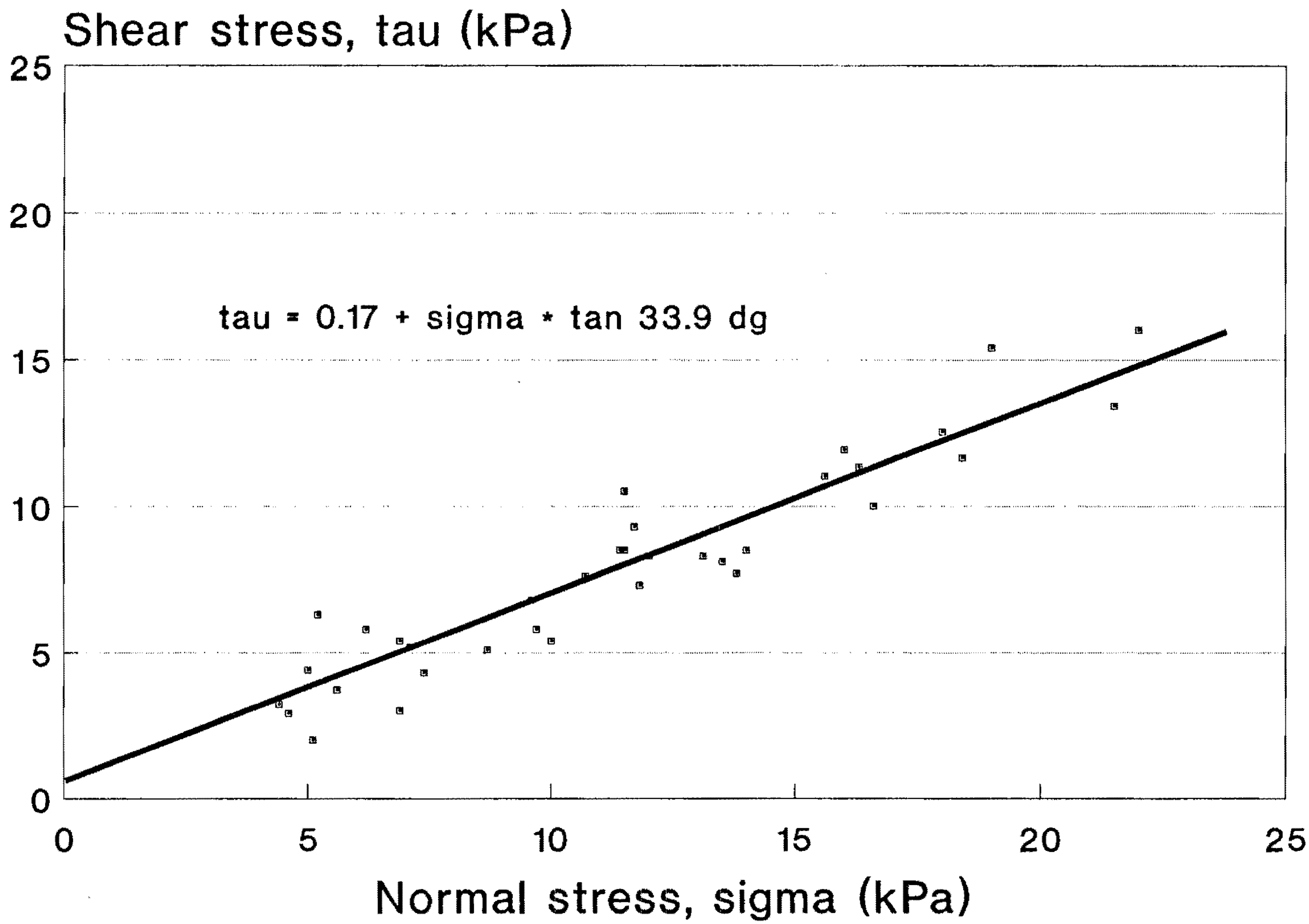

Figure 4. Linear regression plot of triaxial test results.

Headwall Slope Stability Analysis

The soil testing results can be used to make a determination of the factor-of-safety for the headwall study sites. Of interest is the calculated difference in FS for the two soil profile locations in a particular headwall. If the slope under consideration is an order of magnitude longer than the soil is deep, the two-dimensional infinite slope model can be used to determine a factor-of-safety. The model has the form:

$$FS = \frac{c' + c_r - u(\tan \phi')}{\gamma_m h \cos \beta \sin \beta} + \frac{\tan \phi'}{\tan \beta} \qquad (10)$$

where
c_r = root cohesion
γ_m = moist soil unit weight
u = pore water pressure
h = soil depth above failure plane
β = slope of soil mass in degrees

and other terms as previously defined.

A three-dimensional form of this model was derived which considered root strength in the appropriate upper layers of the soil, and evaluated the strength contribution from the sides of the assumed failure block, as well as the base. The results of the factor-of-safety calculations for the 3-D case were not unsimilar to those obtained using the two-dimensional form. For purposes of clarity, the two-dimensional model is presented. It is recognized that the validity of the assumptions this analysis requires, and the use of the two-dimensional model for these slopes may not be entirely appropriate, but is thought to be of secondary concern in evaluating differences in calculated results, and not absolute values.

Assumptions concerning the following are required:

1) an average of root cohesion;
2) the location in the profile at which failure occurs'
3) the pore pressure regime within a headwall.

An average root cohesion of 1.68 kPa for swordfern understory similar to that found on the study sites was reported by Burroughs (1984). This value was used in the calculations. It is assumed that failure occurs at the base of the soil profile, therefore, h equal total soil depth. However, the soil depth value used for HW2 was 95 cm, the depth of the active colluvium. A moist soil unit weight corresponding to field conditions at sampling was also used.

In order to determine the differences in stability for a winter-time condition when failures typically occur, a FS of 1 was assumed for one site within a headwall, and u back-calculated. This value of u was then used in determination of the FS for the second site within that headwall. This method ensures that the FS comparison is made for the appropriate time, when some region of the slope is at limit equilibrium.

Table 4 shows the results of this procedure for the five headwalls for which strength parameters were determined. Results are expressed as the difference between the calculated FS and the assumed FS within a headwall, as a percent of the assumed FS. In equation form, the expression is:

$$\Delta FS = \frac{FS_{calculated} - FS_{assumed}}{FS_{assumed}} \times 100 \qquad (11)$$

Table 4. Percent difference in factor-of-safety (FS) for the slope, from the two soil profiles within each headwall.

Site	ΔFS (%)
HW1	5
HW2	28
HW3	9
HW4	12
HW5	-
HW6	4

The values of ΔFS range from 4% to 28% for the study sites. The largest differences are associated with sites having the greatest differences in soil cohesion values, and soil depth.

The results suggest that in several cases, a different picture of stability emerges for the two different locations in a particular headwall. While simulation of stability model parameters in a Monte Carlo type fashion will not necessarily produce a "wrong" answer regarding slope stability, a "right" answer is dependent on accurate representation of the central tendency and dispersion of the parameters for the hillslope being considered. A statistically sound sample size would be necessary in order to satisfy this requirement.

The model (Equation 10) can be shown to be highly sensitive to changes in cohesion, c', all else remaining constant. Soil depth, h, was also a highly significant variable in a sensitivity analysis of the model parameters. While soil depth is relatively easy determine, the soil cohesion, including root cohesion, is not so readily determined. The importance of obtaining accurate values for these parameters for slope stability analysis cannot be overstated; yet it remains uncertain whether this accuracy can be obtained with current practices. Few field test procedures designed to determine strength parameters of the soil-root system are appropriate for use in steep forested terrain. Additional effort in this area seems warranted.

Discussion

With the exception of the absence of overstory species on the headwalls, the sites sampled exhibited the topography, geomorphology, vegetative, and soil characteristics considered to be typical of headwalls in the Oregon coast range. To some degree, this is by design. As put forth in the criteria for selection, the headwalls studied had certain features which were thought to put them in a high risk group for potential instability. While the results of the vegetative and soil investigations may not be independent of the selection criteria, they were not biased by them.

With respect to the findings of the soil investigations, the soil mantles are thin, slightly- to non-cohesive, silt-sand-gravel mixtures, which have exceptionally low unit weights, and correspondingly high void ratios. The strength parameters determined appear to be within the range of reported results for similar soils (Yee 1975, Alto 1982), but may reflect difficulties in testing of relatively undisturbed soil samples.

Index and Classification Properties

The index properties and classification results, as well as soil strength and slope stability considerations are discussed below.

Specific Gravity, Soil Density, and Unit Weight

Specific gravity of the materials had a mean value of 2.70 (s=0.02). The specific gravity was used to determine additional soil conditions, including void ratio, e. Void ratio is defined as the ratio of volume of voids to volume of solids in an element of soil. Void ratio can be determined from a formula derived from basic phase relationships, utilizing specific gravity and dry soil density. Void ratio is related to percent pore space, n, by the formula:

$$n = \frac{e}{1 + e} \times 100 \qquad (12)$$

The soils exhibit high porosity as was expected from field observations of pedon structure and aggregation. Porosity plays an important role in infiltration and movement of water, and in coarse-textured forest soils is typically highly correlated with the saturated hydraulic conductivity of the soil. High porosities may result from a high degree of aggregation, high organic matter content, colluvial deposition processes - namely mixing of the deposits - and an active soil flora and fauna. As discussed in Testing Methods, organic matter was removed from soil samples used in determination of G_s. These values were subsequently used in calculating the void ratios reported. In order to quantify the effect of the organic matter on the calculation, several G_s values obtained from soils containing organics were used to determine void ratio. Expressed as a percent of e without organics, the difference in e without organics and with organics ranged from +2% to +5% .

Dry soil density and unit weight averaged 0.99 Mg/m^3 (s=0.125 Mg/m^3) and 9.75 kN/m^3 (s=1.23 kN/m^3), respectively. These are considered to be low values for subsoils. The shallow soil depths, and factors discussed previously contributing to high porosity, are thought to explain the relatively low densities. In particular, the processes of colluvial deposition, may be largely responsible for low densities. In addition, this finding may result from the testing methodology. The use of the nuclear density gage differs from more traditional density determination methods in being non-destructive, and is thought to more accurately measure soil density.

Alto (1983) found an average soil density of 1.18 Mg/m^3 (s=0.05 Mg/m^3) for the Mapleton sites sampled. Yee (1975) found dry density to average 1.13 Mg/m^3 (s=0.14 Mg/m^3) for the sandstone derived Bohannon soil.

Atterberg Limits and Clay Mineralogy

The results of the Atterberg Limits tests suggest the soils contain silty, nonplastic or low plasticity fines. Plasticity indices were all relatively low; this indicates a very narrow range of water contents over which the soils behave as a plastic solid. Average plasticity index was 9.2 (s=2.7). Alto (1983) and Yee (1975) obtained similar results. All samples tested by Yee were nonplastic. Of the five Mapleton sites tested by Alto, two soils were nonplastic and the remaining three had low plasticity.

When sampled, a number of the soils had natural water contents which were at or above the plastic limit. Soil HW2-2 had a natural water content higher than both the plastic limit and the liquid limit determined for the soil. Calculations revealed that the water content at saturation of each of these soils is higher than the corresponding liquid limit value. Stable <u>in situ</u> soil structure, resulting from particle aggregation and cementation, may prevent the soil from exhibiting viscous liquid behavior predicted by the liquid limit test when saturated. Remolding of the soil in order to perform the Atterberg tests destroys the natural structure and changes density and other properties of the soil. In addition, only material smaller than the No. 40 sieve size was used in the Atterberg tests. Exclusion of the full range of particle sizes also affects the structural integrity of the soil. The implications of these observations for slope stability are unclear.

Based on the Atterberg test results, the clay fraction (< 2 μm) is thought to account for a low percentage of the soil matrix. The "openness" of the soils, and the amount and rate of water movement likely to occur through them, may result in a high degree of clay elluviation. However, the nature and behavior of the clay fraction may have implications for slope stability, and some attempt to describe the clays likely to be present is warranted.

Istok (1981) investigated the relationship between clay mineralogy and landscape instability in Oregon Coast Range. Clay fractions were characterized by X-Ray Diffraction, Differential Thermal Analysis, or Transmission Electron Microscopy. Clay fractions collected from four debris avalanche deposits which developed on sandstone of the Tyee Formation were analyzed. The fractions consisted primarily of non-expanding layer silicates having large particle size and low water holding capacity. These included chloritic intergrade, dehydrated halloysite, and small amounts of mica. In addition, varying amounts of amorphous gel material was present. Large particle size, small water holding capacity, and small surface charge of the major clay types present in these soils would suggest low intrinsic particle cohesion.

Results of a clay mineralogy identification procedure using a modified Casagrande plasticity chart (Holtz and Kovacs, 1981) and the Atterberg test results, suggest that the most common clay minerals in the test soils are chlorite and halloysite. Figure 5 shows the location of the test soils on the modified plasticity chart in relation to some common soil clay minerals. The work of Istok (1981) would seem to confirm the presence of chlorites and halloysites in soils of Coast Range slopes which are thought to be debris avalanche-prone.

Grain Size Distribution

Results of the mechanical grain size analysis, together with the Atterberg tests, show the soils to be predominantly silt-sand-gravel mixtures. Nine of the twelve soils are classified as SM in the USCS. One soil (HW5-2) was determined to have a greater proportion of gravel size particles which resulted in a GM classification. The remaining two soils (HW1-2, HW2-1) require a dual symbol classification, SP-SM, because of low fines content. These soils are poorly graded sands with nonplastic silts. The particle size distribution suggests these two soils are gap-graded, lacking certain size classes of material altogether.

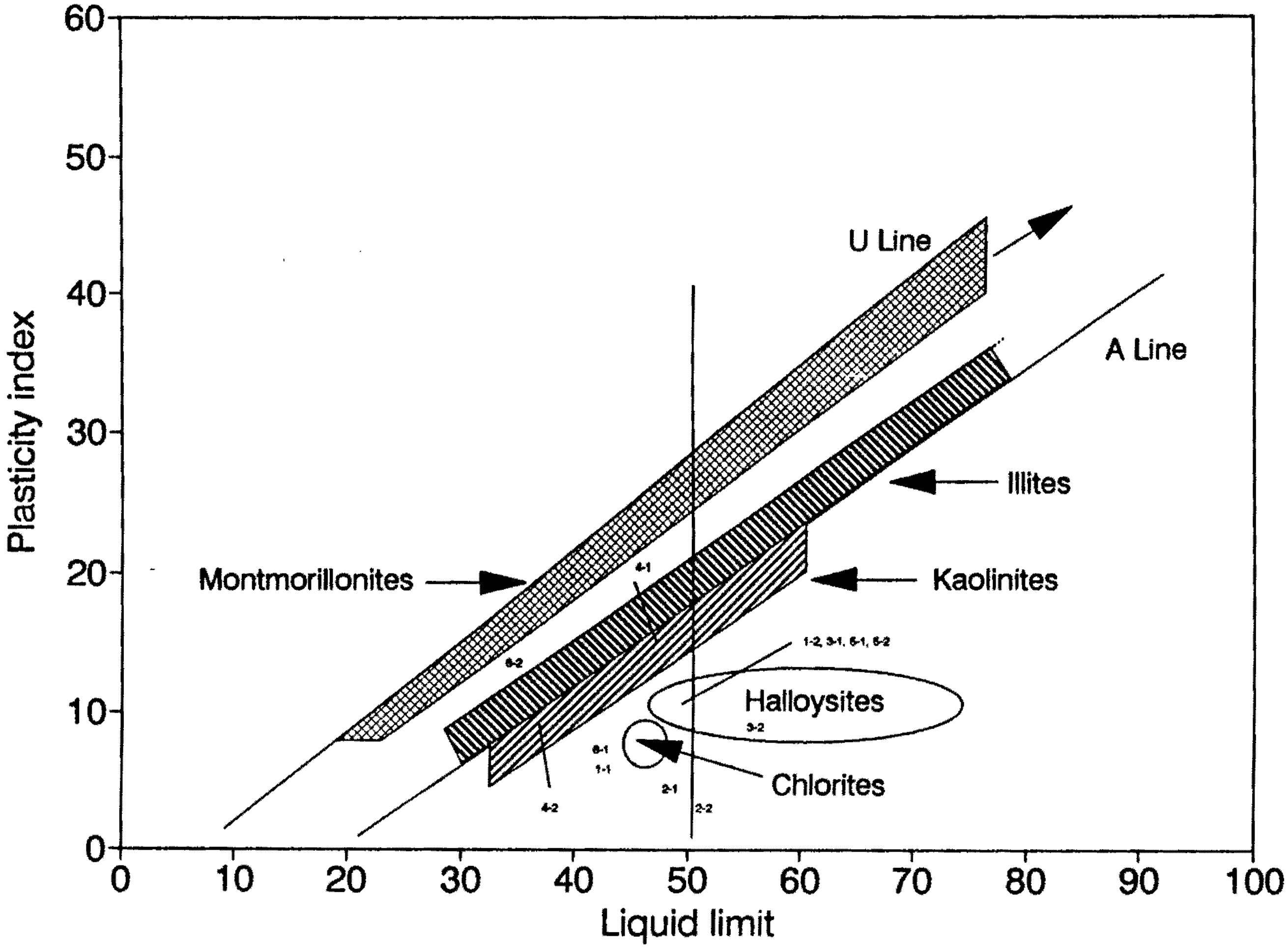

Figure 5. Modified plasticity chart showing location of test soils in relation to various common soil clay minerals.

Effective Strength Parameters

The regression analysis of soil strength values determined in this investigation yields a Mohr-Coulomb failure equation of the form:

$$\tau_{ff} = 0.17 + \sigma'_{ff}(\tan 33.9°) \qquad (12)$$

where τ_{ff} and σ_{ff} represent the shear strength and the normal stress on the failure plane at failure, respectively. The coefficient of determination for the regression is 89%. The regression equation has a greater slope ($\tan \phi'$) and lower intercept (c') than the mean values of ϕ' and c' from the eleven test soils.

The Mohr failure hypothesis states that a material will fail when the shear stress on the failure plane reaches some unique function of the normal stress on the failure plane. The regression line in Figure 4 defines the average of that unique functional relationship for the population of test specimens. It is apparent from the hypothesis, however, that no combination of normal stress and shear stress above the true failure envelope can exist, as the material would have failed previously. Therefore it is important to bear in mind that the failure envelope shown (i.e. the regression equation) is the result of a statistical procedure, and is not altogether founded in the mechanics of slope stability processes.

The regression plot reveals a series of three points forming a straight line parallel to, and above the regression line. These three points represent the test results of soil HW2-1, the soil having the highest value of cohesion determined. Interestingly, this soil was one of two requiring a dual classification symbol representing the absence of fines. The soil also had the highest recorded density and unit weight (Table 2). As mentioned, this headwall was unique in the presence of two very distinct soil horizons. At the base was a reddish layer which appeared to be stable and weathering in place. Above this layer was an apparent "active" colluvial layer. The test results reported are from the colluvial soil layer. HW2-2 was similar, having the same two horizons present. The value of c' was also relatively high. One additional feature of HW2 was the high degree of wetness of the soils when sampled. The natural water contents at sampling were quite high (Table 1), and flowing water was observed in the subsoil during sampling. For the above mentioned reasons, the soils of HW2 are considered to be quite different than the other headwalls sampled. However, there was no apparent difference in vegetation composition on the headwall.

The arithmetic mean value of ϕ' for all soils tested was 31.1° (s=2.1°). The average value of c' was 0.67 kPa (s=0.85 kPa). The regression results suggest that there is a high probability (α = 0.01) that the true value of c' is zero. The factor-of-safety calculations for these slopes suggest, however, that even small values of c' are critical to maintaining stability of the sloping soil masses.

It is frequently assumed that a correlation between c' and ϕ' for a soil exists, such that as c' increases the corresponding value of ϕ' decreases. Figure 6 shows a scatter plot of the data from Table 3. No apparent relationship exists between c' and ϕ' for the soils tested.

There is no significant difference in mean values of ϕ' or c' between the Siuslaw, and the Smith-Umpqua Block headwalls tested, as determined by variance ratio test (α=0.01). The soils of the Smith-Umpqua Block do exhibit slightly greater variability of strength parameters between headwalls, however, than those in the Siuslaw region. It appears there is the same degree of variability in both c' and ϕ' values within a particular headwall, as across the whole population of samples. Whether this qualitative observation stems from sampling or testing method, or is natural, is unknown. A more extensive sampling would be necessary to test a hypothesis that these variabilities are equal. However, due to the conditions under which colluvial soils form, namely periodic deposition and mixing, it is not unexpected that the soils exhibit high variability of properties within a headwall.

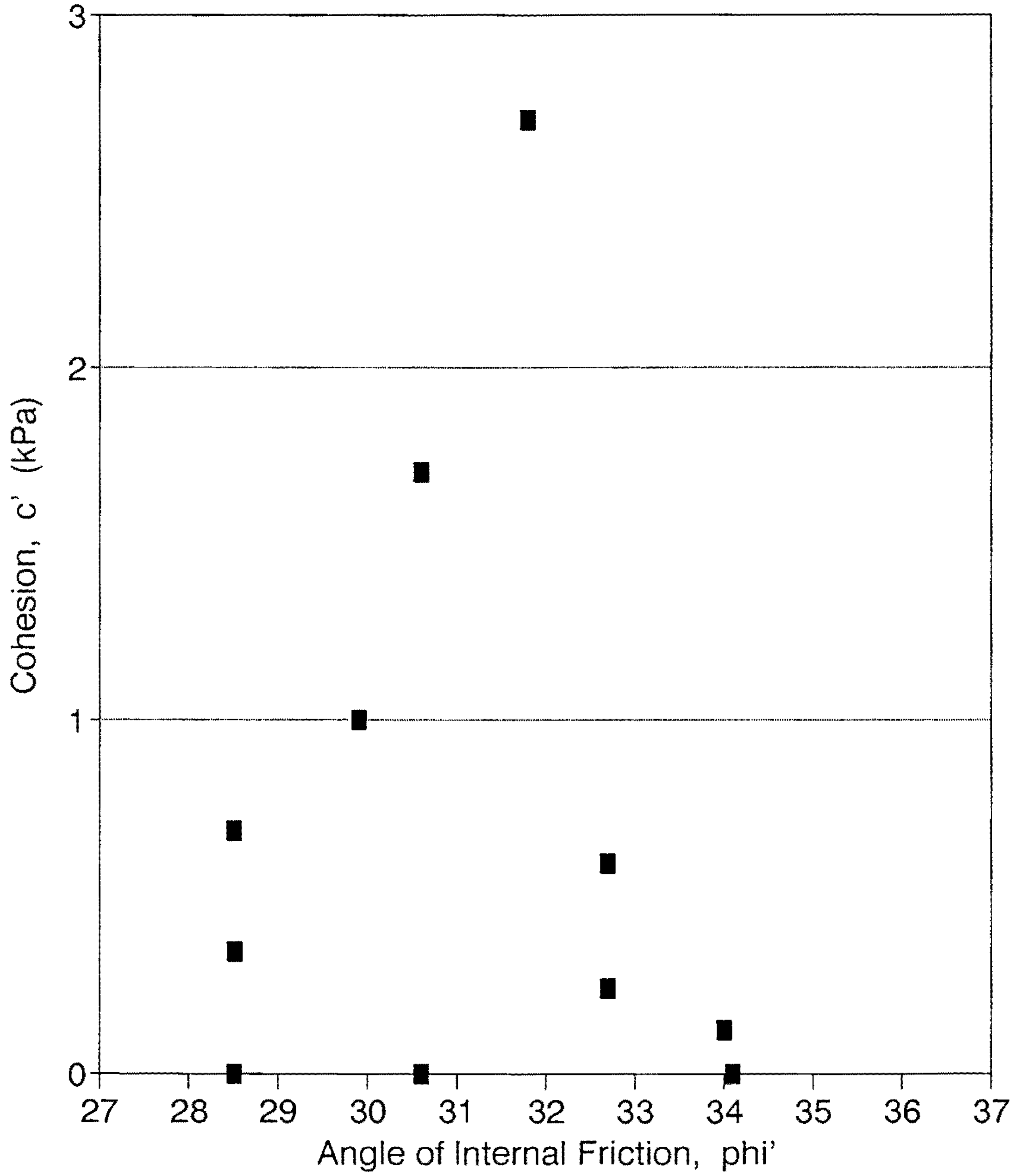

Figure 6. Scatter plot of ϕ' vs. c' for the test soils.

No distinct trends of soil properties or strength parameters within a headwall emerge from these data. Further, there does not seem to be any simple correlation between the index properties and the strength parameters of these soils. In addition, no positive correlation appears to exist between headwall slope (given in Appendix A) and ϕ' for the soils tested.

The strength test results determined here compare closely with those of Yee (1975) for the sandstone derived Bohannon soil. Yee found the average ϕ' to be 29.9° for tests of saturated samples. The soils were all considered to be cohesionless. The undisturbed soils from the Mapleton District tested by Alto (1982) yielded somewhat higher values; the average ϕ' was 37.8°, and the soils also had very little cohesion. Neither of these studies tested soils specifically from headwalls. In addition, the sampling criteria of Alto (1982) explicitly state that soils having large particles, and/or significant vegetative cover were avoided. This suggests that the test results may not be representative of general hillslope conditions in the central Oregon coast range.

In general, the soils tested here are considered to be fairly representative of the major soil types found in the central Oregon coast range. Soils in the headwalls sampled are relatively shallow, averaging just under one meter in depth, and with one exception, appear to be active, homogeneous, colluvial deposits. The soils are typically slightly- to non-cohesive, silt-sand-gravel mixtures, with non-plastic fines. They have low densities and low unit weights, and exhibit high void ratios. The values of density and unit weight for the soils tested here are considered to be very low for subsoils, and such values have not been seen in the literature for otherwise similar forest soils.

Triaxial Testing and Slope Stability Considerations

Testing of undisturbed soil samples from these sites, using the triaxial testing methods employed, proved to be inadequate in some cases. If objects such as large gravel or roots are present in a triaxial test sample, a non-uniform stress distribution may occur. A likely scenario under such circumstances is that the failure plane is forced to occur along the object. Secondly, the failure plane may not form at the angle $45° + (\phi'/2)°$, as is commonly expected in triaxial test samples. As a consequence, the testing results may not be representative of the true values of soil strength parameters. Triaxial testing literature does provide some estimates of the ratio of sample diameter to largest particle diameter for which the value of ϕ' obtained is independent of particle size. Ratios range from 5 to 12 (Bishop and Henkel 1962, Chandler 1973, Tombs 1969). Soil samples from several profiles were retested due to large particles or roots in the initial test. In most cases, the presence of these obstructions could only be determined after completing the test and analyzing the sample. All samples collected from HW5-2 proved too coarse for testing with this apparatus. Referring to Table 1, the USCS classification of this soil was the coarsest of the soils tested. Some unknown bias(es) are likely to exist in the results of the tests, as a consequence of sampling or testing procedures. However, purposeful selection of a sample free of large particles would further bias the testing results for these headwalls, given the coarse nature of the soils. Theoretically, one is not limited in the sample size which can be obtained. There are, however, physical limitations governing the size of a sample which can be tested in a triaxial apparatus. Although triaxial cells ranging to a meter in diameter exist, the apparatus used in conducting the tests reported here had an upper sample size limit of approximately 7.6 cm. The typical sample tested had a diameter of 7.1 cm. This limits the largest particle in the sample to 1.4 cm equivalent diameter for the smallest recommended ratio.

A second difficulty in testing undisturbed samples arose from the occurrence of large voids along the margins of the samples. The presence of these voids resulted in protrusion of the rubber membrane into the voids during consolidation, and uneven distribution of the membrane around the soil. The affect of this on the strength test results is unknown.

The values of ϕ' and c' determined for these soils generally agree with those of similar soils reported in the literature. However, the results of these tests are thought to reflect not only the natural variability resulting from colluvial soil formation processes, but also the inherent difficulties in collecting and testing undisturbed samples. If one assumed the results of the strength tests were accurate, it would seem obvious that some other factors are contributing to the stability of the slopes, given the average gradient of the headwalls is 40.5°. Only one headwall (HW3) showed any evidence of recent debris-avalanche activity; a small slide deposit, with trees approximately 25 years in age growing from the deposit, was observed. Root contribution is a likely source of additional strength. Apparent cohesion resulting from capillary tension in unsaturated soils also increases stability of a sloping soil mass.

Based on these observations it would seem prudent to conclude that the results of these strength tests require validation through additional sampling and testing. The development of more appropriate testing methods, perhaps oriented toward _in situ_ testing would seem warranted.

These considerations raise questions of whether values of strength parameters obtained from standard triaxial tests of undisturbed samples are truly representative of field conditions. Use of these parameters in headwall stability models may lead to erroneous conclusions regarding the failure risk of a particular site. Additional work on characterizing soil strength parameters in headwalls is a necessary condition for further development and use of slope stability models.

Summary and Conclusions

The stability of natural slopes in the steep terrain of the Oregon coastal mountains continues to be a concern of engineers and land use planners. The work reported here is intended to provide information on the vegetative characteristics, and soil strength parameters of a sample of steep, forested headwalls in the Oregon Coast Range, which are pertinent to analysis of slope stability.

Six headwalls were studied for topographic, vegetative and soil characteristics. A summary of the principle findings follows:

1. None of the headwalls studied were found to have live, standing overstory species present on the headwall block, although large roots were present in many of the soil profiles studied.

2. Understory vegetation rooting depth was typically limited to the top 45 cm of the soil profiles, and few small roots were found in subsoils except where soils were very shallow.

3. The soils tested are silt-sand-gravel mixtures with few, nonplastic or low plasticity fines, and are slightly- to non-cohesive.

4. Values of subsoil density and unit weight are considered to be exceptionally low, and are below values reported for similar soils.

5. Sampling and testing of relatively undisturbed soils for strength parameters met with limited success. Large gravel and roots in soil cores is likely to have had an adverse affect on the results of triaxial tests. The direction (+ or -) of the affect on values of strength parameters is unknown.

6. Variability of soil index properties and strength parameters appears to be as great within a headwall as between the headwall study sites.

Several conclusions can be drawn from the investigation. First, while the strength test data are in general agreement with those of similar soils reported in the literature, it is uncertain that a true measure of the soil strength parameters was, or can be obtained, from the methods employed. The difficulties encountered in collecting and testing relatively undisturbed samples, and the exclusion of the root contribution to strength suggests a need to conduct field strength tests which are not limited by particle or root size. Secondly, the variability of soil strength seen within a headwall may be a result of the inherent difficulties in testing discussed. However, the processes of colluvial soil deposit formation suggests that a great deal of variability should be expected.

Based on other studies from the coast range, the soils tested in this investigation are thought to be representative of the major soil types. There does not appear to be any simple correlation between index or classification properties and strength parameters.

Lastly, additional work is needed to further characterize the strength of steeply sloping headwall soils. Given the lack of established overstory trees on the headwalls sampled, and the limited rooting depth of understory species in the soil, the role of roots in providing apparent cohesion to headwall soils is unclear and warrants further study. Further, investigations into the role of subsurface flow routing, particularly macropore flow, would seem to hold promise for advancing the state of knowledge of slope stability processes.

Literature Cited

Alto, J.V. 1982. Engineering Properties of Oregon and Washington Coast Range Soils. M.S. thesis. Oregon State Univ., Corvallis. 61 pp.

Bailey, R.G. 1971. Landslide hazards related to land use planning in Teton National Forest, Northwest Wyoming. USDA, Forest Service, Intermountain Region. 131 pp.

Baldwin, E.M. 1976. Geology of Oregon. Revised ed. Kendall/ Hunt Pub. Co., Dubuque, IA. 147 pp.

Bishop, A.W. and D.J. Henkel. 1962. The Measurement of Soil Properties in the Triaxial Test. Edward Arnold, Pub. Ltd., London, England. 228 pp.

Bishop, D.M. and M.E. Stevens. 1964. Landslides on logged areas in southeast Alaska. USDA, Forest Service, Northern For. Exp. Sta. Res. Pap. NOR-1. 18 pp.

Bowles, J.E. 1978. Engineering Properties of Soils and their Measurement. McGraw-Hill. 2nd ed. 213 pp.

Brown, G.W. 1973. The impact of timber harvest on soil and water resources: Oregon State Univ., Corvallis Ext. Bull. 827. 17 pp.

Burroughs, E.R. 1984. Landslide hazard rating for portions of the Oregon Coast Range. IN Symposium on the Effects of Forest Land Use on Erosion and Slope Stability. Environment and Policy Institute, East-West Center, Univ. of Hawaii, Honolulu. p. 265-274.

Burroughs, E.R., G.R. Chalfant, and M.A. Townsend. 1976. Slope Stability in Road Construction. USDI, Bureau of Land Management, Portland, OR. 102 pp.

Chandler, R.J. 1973. The Inclination of Talus, Arctic Talus Terraces, and Other Slopes Composed of Granular Materials. J. Geol. 81(1):1-14.

Dietrich, W.E. 1989. Headwall Geomorphology in Steep Coast Range Terrain. IN Forestry and Landslides in the Oregon Coast Range. Proc. of a C.O.P.E. workshop held March 7-8, 1989, Newport, OR.

Dietrich, W.E., Wilson, C.J. and S.L. Reneau. 1986. Hollows, colluvium, and landslides in soil-mantled landscapes. IN Hillslope processes, 16th Annual Binghamton Symposia in Geomorphology. London, Allen and Unwin Ltd. Abrahams, A.D. ed.

Dietrich, W.E., and T.Dunne. 1978. Sediment budget for a small catchment in mountainous terrain. Zeitschrift fur Geomorphologie, Suppl. 29. p. 191-206.

Froehlich, H.A. 1989. Headwall Leave Area Effectiveness. IN Forestry and Landslides in the Oregon Coast Range. Proc. of a C.O.P.E. workshop held March 7-8, 1989, Newport, OR.

Fredriksen, R.J. and R.D. Harr. 1979. Soil, Vegetation, and Watershed Management. IN Forest Soils of the Douglas-fir Region. Heilman, P.E., H.W. Anderson, and D.M. Baumgartner eds. Washington State Univ. Cooperative Ext. Serv., Pullman. p. 231-260.

Glock, G.O. 1979. Some Engineering Aspects of Forest Soils. IN Forest Soils of the Douglas-fir Region. Heilman, P.E., H.W. Anderson, and D.M. Baumgartner eds. Washington State Univ. Cooperative Ext. Serv., Pullman. p. 269-277.

Handy, R.L. and N.S. Fox. 1967. A soil bore-hole direct shear test device. Highway Research News. 27:42-51.

Harr, R.D. and C.S. Yee. 1975. Soil and Hydrologic Factors Affecting Stability of Natural Slopes in the Oregon Coast Range. Water Resources Research Institute Report WRRI-33. Oregon State Univ., Corvallis. 204 pp.

Holtz, R.D. and W.D. Kovacs. 1981. An Introduction to Geotechnical Engineering. Prentice-Hall Inc. Pub., Englewood Cliffs, New Jersey. p. 449-458.

Ice, G.G. 1985. Catalog of landslide inventories for the Northwest. National Council of the Paper Industry for Air and Stream Improvement Bull. 456. 78 pp.

Istok, J. 1981. Clay Mineralogy in Relationship to Landscape Instability in the Oregon Coast Range. MS thesis, Oregon State Univ., Corvallis. 95 pp.

Ketcheson, G. and H.A. Froehlich. 1978. Hydrologic Factors and Environmental Impacts of Mass Soil Movements in the Oregon Coast Range. Water Resources Research Institute Report WRRI-56, Oregon State Univ., Corvallis. 94 pp.

Megahan, D.H. and W.F. Gray. 1981. Forest Vegetation Removal and Slope Stability in the Idaho Batholith. USDA, Forest Service, IntMtn For. Ran. Exp. Sta. Gen. Tech. Rep. INT-271 23 pp.

NOAA Climatic Data for Oregon. Monthly and Annual Precipitation Records. U.S. Dept. of Commerce.

O'Laughlin, C.L. 1968. An Investigation of the Stability of the Steepland Forest Soils in the Coast Mountains, Southwest British Columbia. Ph.D. thesis, Univ. of Cantebury, New Zealand. 147 pp.

Patric, J.H. and D.N. Swanston. 1968. Hydrology of slide prone glacial till soil in southeast Alaska. J. For. 66:62-66.

Pierson, T.C. 1977. Factors Controlling Debris-Flow Initiation On Forested Hillslopes in the Oregon Coast Range. Ph.D. thesis, Univ. of Washington, Seattle. 166 pp.

Reneau, S.L. and W.E. Dietrich. 1987. The importance of hollows in debris flow studies: Examples from Marin County, California. IN Debris flows/avalanches; process, recognition, and mitigation. Costa, J.E. and G.F. Wieczorek eds. Reviews in Eng. Geol., Vol. VII:1-26.

Reneau, S.L., W.E. Dietrich, R.I. Dorn, C.R. Berger, and M. Rubin. 1986. Geomorphic and paleoclimatic implications of latest Pleistocene radiocarbon dates from colluvium-mantled hollows, California. Geol. 14:655-658.

Rice, R.M. and J.S. Krammes. 1971. Mass-Wasting Processes in Watershed Management. IN Proceedings of the Symposium on Interdisciplinary Aspects of Watershed Management. Amer. Soc. Civil Eng., Bozeman, Montana. p. 231-259.

Schoenemann, M.R. and M.R. Pyles. 1990. Statistical Description of Triaxial Shear Test Results. Geotech. Testing J., Am. Soc. Testing and Materials. p. 58-62.

Schroeder, W.L. and G.W. Brown. 1984. Debris torrents, precipitation, and roads in two coastal Oregon watersheds. IN Effects of Forest Land Use on Erosion and Slope Stability. Proc. of Sym. held May 7-11, 1984, Honolulu, Hawaii. p. 117-122.

Schroeder, W.L. and J.V. Alto. 1983. Soil Properties for Slope Stability Analysis; Oregon and Washington Coastal Mountains. For. Sci. 29 (4):823-833.

Sidle, R.C. 1987. A dynamic model of slope stability in zero-order basins. IN Erosion and Sedimentation in the Pacific Rim. IAHS Pub. 165. p. 101-110.

Skempton, A.W. 1954. The Pore-Pressure Coefficients A and B. Geotechnique. 4:143-147.

Swanson, F.J., M.M. Swanson, and C. Woods. 1977. Inventory of mass erosion in the Mapleton Ranger District, Siuslaw National Forest. Final Report, Forest Sciences Lab., Corvallis, OR. 62 pp.

Swanston, D.N., R.R. Ziemer, and R.J. Janda. 1983. Influence of Climate on Progressive Hillslope Failure in Redwood Creek Valley, Northwest California. U.S.G.S. Open File Rep. No. 83-259. 49 pp.

Swanston, D.N. 1974. Slope Stability Problems Associated With Timber Harvesting in Mountainous Regions of the Western United States. USDA, Forest Service, PNW For. Ran. Exp. Sta. Gen. Tech. Rep. PNW-21. 14 pp.

Swanston, D.N. 1970. Mechanics of debris avalanching in shallow till soils on southeastern Alaska. USDA, Forest Service, PNW For. Ran. Exp. Sta. Res. Pap. PNW-103. 17 pp.

Terzaghi, K. 1943. Theoretical Soil Mechanics. John Wiley and Sons, New York. 510 pp.

Terzaghi, K. and R.B. Peck. 1967. Soil Mechanics in Engineering Practice. John Wiley and Sons, New York. 2nd ed. 729 pp.

Tombs, S.G. 1969. Strength and Deformation Characteristics of Rockfill. Unpublished Ph.D. Thesis, Univ. London.

Yee, C.S. 1975. Soil and Hydrologic Factors Affecting Stability of Natural Slopes in the Oregon Coast Range. Ph.D. thesis, Oregon State Univ., Corvallis. 204 pp.

Yee, C.S. and R.D. Harr. 1977. Influence of Soil Aggregation on Slope Stability in the Oregon Coast Ranges. Environ. Geol. 1:367-377.

Ziemer, R.R. 1981. Roots and the stability of forested slopes. IN Erosion and Sediment Transport in Pacific Rim Steeplands. IAHS Pub. No. 132. Christchurch, NZ. p. 343-361.

Appendices

Appendix A

Site and Soil Descriptions

<u>Study Site Identification Code:</u> HW1

<u>General Site Location:</u> Hadsall Creek, Siuslaw River region, Road 831, NW¼ S26 T18S R10W W.M.

<u>Elevation:</u> Approximately 305 meters above mean sea level

<u>Topography:</u> Steep, sharply dissected, 80% slope

<u>Aspect:</u> 42°

Soil Profile Description

Soil Profile HW1-1

Depth from soil surface (cm)	Soil description
0-101	dark brown, gravelly sand, dry
101-140	orange-brown, gravelly sand, dry

Soil Profile HW1-2

Depth from soil surface (cm)	Soil description
0-67	dark brown gravelly sand, dry

Study Site Identification Code: HW2

General Site Location: Hoffman Creek, Siuslaw River region, Road 856, NE¼ S19 T18S R10W W.M.

Elevation: Approximately 207 meters above mean sea level

Topography: Steep, sharply dissected, 90% slope

Aspect: 38°

Soil Profile Description

Soil Profile HW2-1

Depth from soil surface (cm)	Soil description
0-95	dark brown gravelly sand, moist
95-152	orange gravelly sand, wet

Soil Profile HW2-2

Depth from soil surface (cm)	Soil description
0-95	dark brown gravelly sand, moist
95-152	orange gravelly sand, wet

Study Site Identification Code: HW3

General Site Location: Karnowsky Creek, Siuslaw River region, Road 867, NW¼ S25 T18S R11W W.M.

Elevation: Approximately 287 meters above mean sea level

Topography: Steep, sharply dissected, 80% slope

Aspect: 334°

Soil Profile Description

Soil Profile HW3-1

Depth from soil surface (cm)	Soil description
0-58	very dark brown gravelly sand, dry

Soil Profile HW3-2

Depth from soil surface (cm)	Soil description
0-46	very dark brown gravelly sand, moist

Study Site Identification Code: HW4

General Site Location: Harvey Creek, Smith-Umpqua Block, Road 41, NW¼ S25 T21S R11W W.M.

Elevation: Approximately 317 meters above mean sea level

Topography: Steep, sharply dissected, 95% slope

Aspect: 150°

Soil Profile Description

Soil Profile HW4-1

Depth from soil surface (cm)	Soil description
0-101	dark brown gravelly sand, moist

Soil Profile HW4-2

Depth from soil surface (cm)	Soil description
0-94	dark brown gravelly sand, dry

Study Site Identification Code: HW5

General Site Location: Harvey Creek, Smith-Umpqua Block, Road 128, NW¼ S25 T21S R11W W.M.

Elevation: Approximately 305 meters above mean sea level

Topography: Steep, sharply dissected, 88% slope

Aspect: 348°

Soil Profile Description

Soil Profile HW5-1

Depth from soil surface (cm)	Soil description
0-64	medium brown, gravelly sand, very moist

Soil Profile HW5-2

Depth from soil surface (cm)	Soil description
0-98	medium brown, very gravelly-sand, very moist

Study Site Identification Code: HW6

General Site Location: Otter Creek, Smith-Umpqua Block, Road 41, NW¼ S27 T21S R11W W.M.

Elevation: Approximately 256 meters above mean sea level

Topography: Steep, sharply dissected, 80% slope

Aspect: 40°

Soil Profile Description

Soil Profile HW6-1

Depth from soil surface (cm)	Soil description
0-116	very dark brown, gravelly-sand, dry

Soil Profile HW6-2

Depth from soil surface (cm)	Soil description
0-79	very dark brown gravelly-sand, moist

Appendix B

Typical Headwall Contour Map
With Vegetation Types and Soil Profile Locations

Headwall Contour Map Legend[a]

Map Symbol	Scientific Name	Common Name
AM	Acer macrophyllum	bigleaf maple
AR	Alnus rubra	red alder
CC	Corylus cornuta	hazel
DF	Pseudotsuga menziesii	Douglas-fir
GS	Gaultheria shallon	salal
OS	Holodiscus discolor	ocean spray
RM	Rhododendron macrophyllum	rhododendron
SB	Rubus spectabilis	salmonberry
SC	Sambucus callicarpa	red elderberry
SCu	Ribes bracteosum	stink currant
SF	Polystichum munitum	sword fern
TB	Rubus parviflorus	thimbleberry
VM	Acer circinatum	vine maple
VO	Vaccinium ovatum	huckleberry

Additional Map Symbols

Map Symbol	Identification
CP	Headwall Critical Point
Soil Profile 1	Soil Profile 1 Location
Soil Profile 2	Soil Profile 2 Location

(a) Not all vegetation types shown on map. Legend gives all vegetation types mapped on the six headwall study sites.

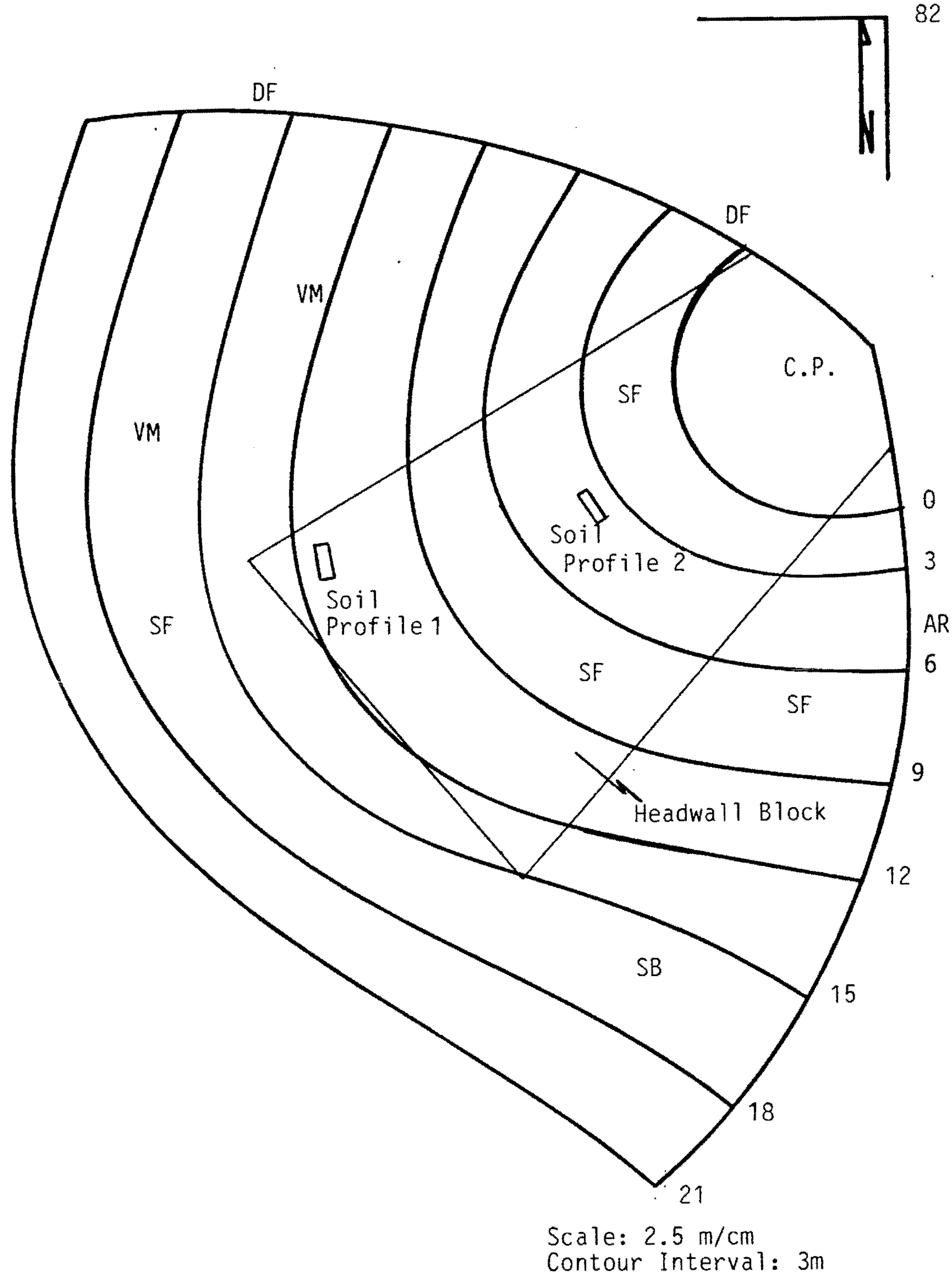

Figure 7. Typical headwall contour map with vegetation types and soil profile locations.