
Volume I
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Cover Photo Caption:

EYESEE DEBRIS SLIDE, Klamath National Forest, Region 5, Yreka, CA

The photo shows the toe of a massive earth flow which is part of a large landslide complex that occupies about one square mile on the west side of the Klamath River, four air miles NNW of the community of Somes Bar, California. The active debris slide is a classic example of a natural slope failure occurring where an inner gorge cuts the toe of a large slump/earthflow complex. This photo point is located at milepost 9.63 on California State Highway 96.

Photo by Gordon Keller, Plumas National Forest, Quincy, CA.

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Volume I

Coordinators:
Rodney W. Prellwitz
Thomas E. Koler
John E. Steward

Editors:
David E. Hall
Michael T. Long
Michael D. Remboldt
Dedication to Rod Prellwitz

Courtney Cloyd, Engineering Geologist, Siuslaw National Forest

The rugged terrain of America's national forests presents engineering geologists and geotechnical engineers with many challenges. Small budgets and staffs encourage innovative exploration techniques, while high design standards are necessary for compliance with Federal resource protection regulations. These sometimes conflicting conditions have fostered the development of skilled professionals, who often rely on the experiences of their colleagues for help with difficult problems. In this spirit of cooperation the Slope Stability Reference Guide was developed, a spirit that characterizes the career of the guide's instigator, Rod Prellwitz. In recognition of his years of guidance and support as a mentor and friend to geotechnical specialists in the Northwest, we dedicate these volumes, a summary of what we have learned together.

Rod Prellwitz's career as a geotechnical engineer is the result of his love of the mountains and forests of the West and his love for the process of solving geotechnical problems. The symbols of these two loves are his continuing enthusiasm for field work and the license plate on his four-wheel-drive pickup, GEOTECH.

Rod's professional training began in 1957 at the Montana School of Mines (now Montana College of Mineral Sciences and Technology), where he studied geological engineering with a petroleum option. After graduating in 1961, he worked for 3 years as a seismic geophysicist in Wyoming, Utah, and Colorado with Texaco, Inc. During that time he realized that career advancement in the petroleum industry meant spending long periods of time in such "God-forsaken places" as Houston, Tulsa, and Denver—a depressing prospect for a young man who loved the big, empty country of the West.

In 1964, Rod heard that the USDA Forest Service was looking for a civil engineer on the Apache National Forest, and soon he was on his way to Springerville, Arizona, leaving the oil business behind forever. In 1965, he moved on to a position in preconstruction engineering in the regional office at Missoula, Montana, seizing a chance to return to the State he had grown to love. By 1966, he had moved on to Hamilton, Montana, and the Bitterroot National Forest, where he eventually became an assistant forest engineer. Rod later found that he was spending far too much time managing the motor pool and not enough time on engineering; so, in 1969 he transferred to the Northern Region Materials Investigation Unit back in Missoula as a geotechnical engineer.

His role as a regional office geotechnical engineer gave him an opportunity to see much of Idaho and Montana and a full range of geotechnical problems. Rod took graduate courses at the University of Idaho in the early 1970's, graduating with an
M.S. in civil engineering with a geotechnical option in 1975. In 1979, he became a research geotechnical engineer at the Intermountain Research Station in Missoula, continuing there until his retirement in early 1993.

Slope stability analysis and ground water monitoring have been Rod's primary professional interests as a researcher. Since 1975, he has written more than a dozen professional papers, made numerous presentations before professional societies on geotechnical topics, and developed several practical guides and handbooks for use by Forest Service field specialists. Among his papers "SSMOS"—Slope Stability Analysis by Three Methods of Slices with the HP-41 Programmable Calculator in 1985 and "SSIS" and "SSCHFS"—Preliminary Slope Stability Analysis with the HP-41 Programmable Calculator in 1988 were instantly popular with Forest Service geotechnical engineers and engineering geologists in the Northern and Pacific Northwest Regions.

Through the development of those publications, the Pacific Southwest and Pacific Northwest Region geotechnical specialists learned what their Northern Region colleagues had known for some years: Rod was a research scientist who understood the field conditions they faced every day, and he was interested in helping them find the answers to their technical problems. He routinely provided technical support to national forest engineers, geologists, and soil scientists who were trying to apply textbook soil mechanics and ground water principles to steep forested terrain conditions and finding that the two were not always compatible.

Rod is probably best known in the Forest Service for developing what is commonly called the three-level slope stability analysis concept. The three-level system grew out of his understanding that engineering geologists and geotechnical engineers could contribute their knowledge of soil mechanics and physical processes to forest planning and decision making. His 1983 presentation to the Transportation Research Board 62nd Annual Meeting, Landslide Analysis Concepts for Management of Forest Lands on Residual and Colluvial Soils (T.R. Howard and W.D. Wilson, co-authors), summarizes the three-level system and sets the stage for his later work on Level I Stability Analysis (LISA), Stability Analysis for Road Access (SARA, the level II program), and XSTABL (the level III program). LISA and XSTABL have become standard analysis tools for resource planning level and site-specific slope stability assessments, respectively, on national forests in the western United States.

As helpful as these tools have been, Rod's willingness to teach others what he has learned is his greatest contribution to the profession. He has always been more than generous with his time when people ask for help, preferring a field trip to look at the problem to talking about it on the phone. His informal style and ever-present sketch pad frequently turn simple questions into impromptu tutoring sessions, where gaining knowledge and understanding are as important as finding the answer to the problem. Over the years, Rod has been a frequent contributor to technical workshops held by the Forest Service, the Oregon and Washington Departments of Transportation, and the Federal Highway Administration. In the last 5 years, he has been a principal instructor at more than half a dozen Forest Service-sponsored workshops focused on the LISA computer model, demonstrating his belief that technical information is most effectively exchanged through discussion and shared experience.

Even though we won't see the familiar Montana GEOTECH license plates as often in our office parking lots, Rod has left us a rich legacy of his knowledge and
experience and a promise to be available occasionally for a look at an interesting problem. In thanks for his many gifts, we dedicate the *Slope Stability Reference Guide* to Rod Prellwitz.

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Dedication

Kendall Neal  
Thomas E. Keller  
Laurie Freeman  
Jordan R. Keller  
Ron C. Patino  
David Wall  
Kenneth W. Baldwin  

Houg Williamson  
Denise Larson  

Andrew E. Giddens  
Richard Van Dyke  
Clifford C. Deming  
Ted S. Steward  
John E. Barden  
John W. Lartigue  
Mike Hamal  
Cindy Ricks  

Cartney Cloyd  
Mark Leuenberger  

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Dedication
Acknowledgments

The authors, editors, and coordinators thank the following individuals for their review efforts and comments that helped to make this publication more user-friendly—quite often sparing the authors some embarrassment. We especially acknowledge Dr. Robert L. Schuster of the U.S. Geological Survey and Peter Jones of the Rogue River National Forest for their extremely detailed reviews. In addition to the internal review by the authors themselves, the following group of professionals made a tremendous contribution:

- Matt Brunengo, Washington Department of Natural Resources
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- Richard Kennedy, Bridger-Teton National Forest
- Allen King, Plumas National Forest
- David Knott, GAI Associates
- Margaret McHugh, Gifford Pinchot National Forest
- Jim McKean, USFS Pacific Southwest Region
- Dan Miller, University of Washington
- Stan Miller, University of Idaho
- Ken Neal, Neal and Associates
- Ann O'Leary, Naval Facilities Command
- Bill Powell, USFS Pacific Northwest Region
- Bruce Vandre, USFS Intermountain Region
- Richard Watanabe, Oregon Department of Transportation
- Chester F. ("Skip") Watts, Radford University.
Slope stability studies in the USDA Forest Service have been conducted successfully in accordance with a three-level concept.

**Level I**

Level I, the most general, is conducted for watershed analyses, ecosystem management support, and timber sale area planning. Level I slope stability studies include reviews of air photo coverage and geologic and geotechnical reports; a brief reconnaissance to verify slope processes and to map soil and rock units and landforms; and development of geomorphic zones based on slope form, soil and rock characteristics, and geologic processes. Each geomorphic zone is assigned soil and rock units for analysis and evaluation for potential failure due to natural processes and forest management activities. The LISA (Level I Stability Analysis) and DLISA (Deterministic Level I Stability Analysis) programs are often used in the process.

**Level II**

Level II, an intermediate level used for evaluation of slope stability along road corridors and other route studies, consists of the office elements of a level I study, a reconnaissance (including sketches of slope characteristics of each alternative transportation corridor), and defining geomorphic and road design segments based on slope, rock, soil, and drainage characteristics and geologic processes. These analyses are often completed using slope stability charts and the DSARA (Deterministic Stability Analysis for Road Access) slope stability program. The probabilistic SARA (Stability Analysis for Road Access) program is still under development.

**Level III**

Level III, the most detailed (site-specific) level, is intended for design of stabilization measures. In addition to the elements identified under level II for the transportation route evaluation, the level III study includes measurement of field-developed cross-sections and installation of site-specific monitoring instrumentation, as appropriate. Level III analyses are often completed using the XSTABL (method of slices) interactive program for soil and the Federal Highway Administration’s rock slope stability analysis method for rock slopes.

In addition to the three-level method, this guide discusses the forest planning process, geomorphology, channel processes and sediment budgets, climatology, soil and rock mechanics, hydrogeology, exploration and testing methods, and risk analysis as they apply to slope stability. The guide also contains sample problems for analysis, remediation design, and specifications for construction.
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SECTION 1

INTRODUCTION AND SUGGESTIONS FOR USE

Principal contributors:

Joe Bailey, Engineering Geologist (Section Leader)
USDA Forest Service
Regional Office Engineering
P.O. Box 3623
Portland, OR 97208

Tom Koler, Research Engineering Geologist
USDA Forest Service
Intermountain Research Station
1221 S. Main
Moscow, ID 83843

Ken Neal, Engineering Geologist
Kenneth Neal & Associates
2014 Baker Terrace
Olympia, WA 98501

Rod Prellwitz, Geotechnical Engineer
USDA Forest Service
Intermountain Research Station
1221 S. Main
Moscow, ID 83843
Section 1. Introduction and Suggestions for Use

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Appendix

1.1 Introduction to the Three-Level Stability Analysis Concept—Chestershire and Backdrop Timber Sales: Case Histories of the Practice of Engineering

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1A. Guide to the Guide

All geotechnical specialists experience, at some time in their careers, the problem of how to provide an appropriate, accurate, understandable answer to a question posed by a land manager or project engineer. The specialist realizes that the manager probably has little knowledge of the scientific complexity involved in answering a seemingly simple geotechnical question. Of several legitimate answers, which is "correct"? The anxiety level rises if a geotechnical mentor is not readily available to guide the specialist through the process of providing the correct answer. Of several possible ways to begin gathering and analyzing data, which is the most valid? The most cost effective? The most truly useful? We have all gone through this experience, and continue to do so. Therefore, we have compiled this slope stability reference guide to help us and those who work with us to better understand how geotechnical specialists get work done. We have included real-world examples of common geotechnical problems, along with the reasons behind, and the methods of, the investigative techniques employed. The purpose of this guide is to provide a practical document that geologists and engineers can use in their work. We hope that your copy will become tattered and dog-eared as you use it to provide these answers.

1A.1 Purpose of the Guide

This guide can be viewed either as a textbook describing concepts and theories or as a laboratory workbook with practical problems. Each section contains both theory and practical examples; we strongly recommend that both be understood and used. A knowledge of the theory behind the methods employed in a particular example helps you to recognize the differences between that example and your specific real-world problem. However, knowing the theories will not help much unless you recognize which are relevant to your particular problem.

Section 1 contains the introduction to the guide and an introduction to the three-level stability analysis concept.

Section 2 discusses the role of stability analysis in cumulative effects analysis and empirical analysis of an area.

Sections 3 and 4 describe the concepts central to the analysis of slope stability affecting both natural and constructed slopes.

Sections 5 and 6 give detailed presentations on application of the three-level system to typical field problems. Section 5 emphasizes analysis of slope stability, while section 6 emphasizes methods of stabilization.
Figure 1A.1 is an idealized three-level slope stability analysis flowchart. This chart will appear in the introduction to each section with the area of analysis discussed in that section highlighted.

![Slope stability analysis flowchart](image)

*Figure 1A.1.—Slope stability analysis flowchart.*

We hope you find the guide to be useful. If you find portions of it unclear, debatable, or in error, we would like to hear from you. Names and addresses of the authors for each section are listed at the beginning of the section.

### 1A.3 Suggested Use

#### 1A.3.1 Land Managers

The first two sections of the guide will serve land managers’ needs by introducing them to the three-level system and its use in resource planning. The remainder of section 1 describes the three-level system, its general methods of analysis, and its limitations. In section 2 we discuss the use of slope stability assessment in resource planning. After reviewing these two sections, the manager should be able to communicate better with—and understand the degree of confidence communicated by—the investigator.

The details of investigative and analysis techniques (sections 3–5) may not be of interest to the land manager, but the methods of stabilization in section 6 may help in understanding alternative solutions.
1A.3.2 Geologists/Geotechnical Engineers

Geologists and engineering geologists will have use for the entire design guide, but will probably find the techniques for level I and II analyses the most useful. These two levels, for reconnaissance and project planning, respectively, describe a method of predicting slope instability over large areas.

Reconnaissance-level assessment refers to the analysis of large areas for relative landslide-hazard potential. This level of analysis is used for input to forest planning, timber sale and resource allocations, environmental assessment reports, and transportation planning. The LISA (Level I Stability Analysis Ver. 2.0 1991) computer model discussed in the guide can be used to delineate areas susceptible to landslides on a broad scale to alert land managers to those areas where the risk is greatest (Hammond et al., 1992).

Project planning refers to the analysis of designs proposed to implement a management objective. For example, a proposed road corridor is analyzed for preconstruction and postconstruction slope stability. This also includes a sensitivity analysis for individual proposed road design segments that cross potentially unstable areas identified in LISA. DLISA (Deterministic Level I Stability Analysis Ver. 1.02 1991) works well for sensitivity analysis. An evaluation of road prism conditions (e.g., cut slope heights and angles, fill slope angles and depths, fill compaction, and drainage) can be completed with the results from the sensitivity analysis. A level II computer program, SARA (Stability Analysis for Road Access), is being developed and DSARA (Deterministic Stability Analysis for Road Access) is being perfected at the Intermountain Research Station.

Experienced geologists and engineering geologists will find level I and II analyses useful when a proposed project may change soil physical properties, slope configuration, or surface and subsurface water flow. Experienced engineering geologists understand the basic concepts contained in the guide. Those geologists with little experience in the engineering arena may not be familiar with the analytical techniques used. Those freshly out of school with degrees in classical geology should seek additional training to gain a working knowledge in soil mechanics, strength of materials, and basic road design. For the less experienced practitioner, a mentor assisting and verifying input data will help ensure accurate results.

1A.3.3 Geotechnical Engineers

Geotechnical engineers will have use for the entire guide, but will probably find the sections explaining design applications for level II (project planning analysis) and level III (site-specific analysis) the most useful. During a project design, efforts are generally made to minimize cost, which usually translates into the minimum functional design. However, site constraints or aesthetic considerations may require a deviation from the standard textbook procedure. This guide shows how to calculate factors of safety for these minimal or unique solutions. It is also useful in answering design questions where standard designs are inappropriate.

During failure analysis, the guide is useful in leading one through the back-calculation process. By back-calculation, the failure parameters can be deduced. Designing a repair is then a matter of bringing the factor of safety up to a desired level by buttressing, drainage, or other means. Usually, several alternatives for failure correction are possible. The guide shows ways to analyze these alternative repairs and calculate their factors of safety. Coupled with cost data, these alternatives can be presented to management.
Geotechnical engineers understand the concepts and analysis techniques contained in the guide. Most civil engineers are familiar with these methods. Those who have not worked a number of these problems in the past may want a peer review of their methods and assumptions.

1A.3.4 Non-Geotechnical Specialists

Personnel without formal geotechnical training—such as many hydrologists, soil scientists, foresters, and other specialists—may use the guide to gain an understanding of the types of slope stability problems that geotechnical personnel are able to analyze and solve. Non-geotechnical specialists having extensive exposure to and experience with the underlying concepts and analytical techniques of the guide will find it useful in the understanding of slope stability problems.

The reader must bear in mind that an understanding of the slope-movement processes being modeled, as well as how the model represents the processes, is needed. The model will produce an output regardless of input inaccuracies. The guide is not a substitute for a thorough understanding of slope stability concepts; the guide in itself will not make anyone an expert.
1B. Introduction to the Three-Level Stability Analysis Concept

Slope stability specialists must be able to predict how land management activities—such as logging operations and road construction—will impact land stability. This assessment is useful to the land manager, usually a district ranger, in evaluating the level of the potential watershed impacts among various alternatives. The land manager usually has no professional background in evaluating slope stability and therefore must rely on the knowledge, skills, and abilities of the district earth scientists and engineers or specialists from a zone office or forest headquarters. This predictive capability is the fundamental principle of the three-level stability analysis concept. By using this method the specialist will be successful in assisting the decision maker.

Determining which technique to use can be confusing for the specialist who does not routinely perform slope stability analyses. Effective communication between the specialist and the person requesting the analysis can minimize this confusion. With effective communication there is an understanding of the basic reasons for the work. Effective communication also leads to a slope stability analysis matched to the level and complexity of the problem and avoids analysis simply for its own sake. The common result of ineffective communication and inappropriate analysis is that the requester will not ask for this service in the future.

It is important for the specialist to understand that the data gathered will be used to define slope conditions. Moreover, he or she should know that the analysis is being made to model the current land stability conditions and potential impacts as a result of excavation, logging, construction, damming, or other management activities. Also, the engineer or scientist should know how the results fit into a rational decision-making framework. Considering the wide range of land management decisions requiring a supporting slope stability evaluation, it is important that the prudent specialist adapt the investigation and analysis to the decision being made.

Many forested lands in the western United States have the potential to be unstable because of geologic and hydrologic conditions and material strength properties. The National Environmental Policy Act (NEPA) of 1969 and the National Forest Management Act (NFMA) of 1976 require that the management of these lands includes stability assessment at all planning levels: resource allocation (forest plan, 10-year scheduling, and timber sale planning), project planning (timber sale environmental assessment and transportation planning), and project development and monitoring.
(site stabilization and implementation of the preferred alternative). This requirement has led to the development of the three-level stability analysis concept (Prellwitz et al., 1983; Prellwitz, 1985). Figure 1B.1 is a flowchart of this analysis system.

The three levels are progressive in scale, detail, complexity, and accuracy. This progression focuses attention on specific problems in a logical order. It also allows the analyzer to draw from the previous analysis and data bases as new data are developed to support the higher analysis levels. At levels I and II, the analysis techniques have a probabilistic version to allow for the determination of parametric values over the areal extent of the analysis. Also, the probabilistic output lends itself well to the decision-making process. A comparison of the combination of slope failure probability with the relative failure consequences eases this process, allowing the manager to base the decision on acceptable risk levels. The deterministic version of the analysis is still necessary to facilitate sensitivity analysis, parametric value evaluation, and back-analysis.

The following levels ensure that suitable investigation and analysis techniques are available for the qualified specialist to provide timely and efficient input to support decisions:

**1B.2.1 Level I: Resource Allocation**

The types of decisions at this level are made from data gathered at a watershed scale (usually 1:24,000 or greater), and the evaluation of the relative stability assesses hazards involved in the proposed activity. At this level the decision maker determines the resource allocations and where to develop them. The stability assessment usually uses available inventory data in the simplified stability analysis techniques. In most cases, these data are from a geographic information system (GIS), a geologic
resource and conditions data base, or both. Because the stability of the entire land mass is being considered, the analysis techniques must model the failure mode of natural slopes. These failures may be either planar (translational) or circular (rotational) in shape, and the analysis must be able to model both modes. The stability of constructed slopes usually is not evaluated at this level.

1B.2.2 Level II: Project Planning

The next major level of decision making requiring slope stability input occurs in the project planning phase (e.g., timber sale environmental assessment, transportation planning, and road closure planning). Timber-harvest decisions on tree removal and the impacts on the stability of natural slopes include the modeling of tree-removal effects. The two important factors in this analysis are root strength and ground water. The stability of constructed slopes is often more significant than that of natural slopes at the project level; however, a simplistic analysis that estimates the relative stability of the road slope is adequate at this level. Critical locations identified at this level will be considered for level III analysis. A primary difference between the level II and level III analyses is that the latter must allow the analyzer to evaluate many unique site conditions and stabilization measures. The data base used in the level II analysis must be more accurate than that used in level I. Project reconnaissance includes field-developed cross-sections measured at locations critical for this level. Measurements of the field-developed cross-sections are usually at a 1:600 to 1:3,600 scale; therefore, the data collected will be more accurate than that collected for level I, but less accurate than that collected for level III.

1B.2.3 Level III: Site Stabilization

Critical sites recognized during the level II analysis may require a level III analysis if the site cannot be avoided. For example, to gain access to a landing, the district transportation planner lays a proposed road corridor across an inactive landslide. For economic or ecological reasons the district ranger chooses this route. The level II analysis predicts slope stability problems will occur. The next step, then, is to complete a level III analysis to evaluate various stabilization methods—such as buttresses, drainage, retaining walls, or additional excavation—for the site. All options—including the do-nothing alternative—are available for consideration so that the manager can have a perspective of the inherent cost and resource risk of each. The report from this analysis allows the decision maker to compare possible stabilization alternatives and to make a decision based upon relative cost and resource risk.

Section 6 of this guide includes a detailed description of these analyses. Measurements for site stabilization investigations are usually at a 1:120 to 1:600 scale. These measurements and subsequent analysis techniques are more complex than those for levels I and II. This increase in data resolution is necessary to determine existing conditions and to evaluate all possible stabilization alternatives.

1B.3 Stability Analysis Applicable at the Three Levels

There is a danger in fixing a certain analysis method to a certain decision level because there is some overlap in applicability. Consideration of the type of slope (natural or constructed) and the failure mode (translational or rotational) is always important. In a level I decision, stability of large land masses is of primary concern, whereas in level III decisions, stability of specific critical slopes (usually constructed) is the primary focus. In level II the decisions are somewhere between these two levels. The following analysis techniques are the most applicable:
1B.3.1 Level I: Infinite Slope Equation

An explanation of this simple and versatile equation is presented in section 5 of this guide. Although simple, it still does an adequate job of allowing one to model all of the important factors in analyzing the potential translational failures of natural slopes. In addition, the analysis is applicable to rotational characteristics (Prellwitz, 1975; Ristau, 1988). Together with a Monte Carlo simulation technique, this equation is a powerful tool for incorporating spatial variability of input parametric values and the uncertainty of the analyzer in determining what those values are (Hammond et al., 1992).

1B.3.2 Level II: Stability Number Solutions

The infinite slope equation is still applicable to evaluate the impacts of timber harvest on the stability of natural slopes. The analyzer can anticipate changes in two important factors in his or her modeling that might result from the removal of trees: root strength and ground water in response to precipitation changes. To evaluate the relative stability of proposed constructed road cut-and-fill slopes on the natural slopes, two stability number solutions are available:

Critical Height Analysis

A critical height analysis predicts the maximum (critical) height for a stable constructed slope. A computer program using correction factors for subsurface conditions with Chen and Giger’s method (1971) is currently under development.

Estimated Factor of Safety

This type of analysis predicts the factor of safety against failure that one might expect from a level III analysis of a simple constructed slope. This type of analysis lends itself well to a probabilistic approach using a Monte Carlo simulation similar to the one used for the infinite slope equation. This analysis method may be applicable to supporting level I decisions when the potential of road-related failures is a primary concern. A computer program using subsurface correction factors coupled with the Cousins method (Cousins, 1978) is in development.

1B.3.3 Level III: Method of Slices

At this level of decision making, the specialist must be able to model many unique critical site conditions and stabilization measure characteristics. This requires a more complex analysis technique than those used in levels I and II. The most common technique is to divide the potential failure mass into a series of slices and model the individual slice forces—first individually and then collectively—in a summation process to arrive at a prediction of the overall potential for failure. Several "method-of-slices" solution techniques are available, all having a common soil mechanics base but differing in the manner in which they are satisfied, the more complex and more difficult the analysis becomes. The computer program XSTABL (XSTABL Ver. 4.1 1992) developed by Sharma for Level III analysis uses the simplified Bishop method for rotational failures and the simplified Janbu method for failures of any general shape. These methods are sufficiently accurate for most forest road applications. Sharma (1992) gives a comparison of these methods to other methods of slices.
References


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**Level I Stability Analysis Ver. 2.0 (LISA 2.0).** January 1991. Moscow, ID: USDA Forest Service, Intermountain Research Station, Engineering Technology.


References

APPENDIX 1.1
1.1 Introduction to the Three-Level Stability Analysis Concept

Thomas E. Koler, Research Engineering Geologist, Intermountain Research Station
Kenneth G. Neal, Kenneth Neal & Associates

The following paper gives examples of the three-level analysis concept as used on steep, forested terrain in the Olympic National Forest. The paper also illustrates how slope stability analysis is initiated within the planning process (level I), evolving on a continuum through the project level (level II) and design level (level III). The most important point is that information used in each level can be used in the next levels and vice versa. For example, the design level analyses for road construction and reconstruction in the Binder and Chestershire timber sales were used in the planning analysis of the Backdrop timber sale. Planning boundaries for proposed timber sales frequently change as issues, concerns, and opportunities evolve in the NEPA process. Note that all data available, regardless of artificial boundaries, are viable bits of information at all three levels.

INTRODUCTION

Background

The practice of engineering geology in the Olympic National Forest (Figure 1), as in other national forests, is a recent development resulting from environmental policy legislation enacted in the late 1960s and early 1970s. These laws include the National Environmental Policy Act of 1969 (NEPA), the Federal Water Pollution Control Act Amendments of 1972, the Endangered Species Act of 1973, the Safe Drinking Water Act of 1974, the Forest and Rangeland Renewable Resources Planning Act of 1974 (RPA), and the National Forest Management Act of 1976 (NFMA). Prior to this suite of legislation, the application of earth sciences in the Olympic and most other national forests was primarily the concern of soil scientists. Geologists were involved almost solely with economic mineral deposits. By 1974, engineering geologists, along with geotechnical engineers, technicians, and drill crews, were employed full-time as members of geotechnical teams in many national forests, including the Olympic. Initially, the practice of engineering geology was limited to solving problems related to road design and construction. However, it soon became apparent that the practice of engineering geology had much broader applications, particularly as related to timber sale project planning. The two case histories presented in this paper, Chestershire and Backdrop timber sales, are examples of how our profession has contributed to project work in the Olympic National Forest and how the "state of the art" has progressed since 1975.

Levels of Project Investigation and Application

Since 1975, a majority of project work completed by engineering geologists in the Olympic National Forest has been directed toward planning, locating, designing, and constructing low-volume, single-lane logging roads. Most of this work involves both evaluation and analysis of slope and foundation stability and evaluation of the suitability of rock and aggregate materials for use in construction.

Engineering geology projects in the Olympic National Forest are three-tiered and scale dependent. The theory behind this system is that each tier forms a base on which the next higher tier is built. Level 1, the lowest tier, consists of reconnaissance mapping for area planning purposes, at a scale of 1 in. = 2,000 ft (1:24,000). Level 2, the intermediate tier, consists of reconnaissance mapping for transportation planning, or road or facilities location, at a scale of 1 in. = 300 ft (1:3,600). Level 3, the highest tier, consists of site-specific investigation, usually for design of roads or other facilities, with data collected at a scale of 1 in. = 50 ft (1:600) or less.

The level of data collected is scale dependent. The number of data points and the accuracy and reliability of data collected increase in direct proportion to the square of the percent increase in scale; for example, a section developed at 1 in. = 10 ft has 25 times as many data points as one developed at 1 in. = 50 ft. In like fashion, the type of decision that can be based on a specific set of data is also scale dependent, from Level 1 decisions regarding the suitability of a parcel of land for management, modification, or use, to Level 2 decisions regarding impacts along alternative corridors through that parcel, to Level 3 decisions regarding designs to fit terrain characteristics at a specific site along the selected route (Figure 2).

Geologic Resources and Conditions Database

All project data collected, regardless of level, are entered into the Olympic National Forest Geologic Resources and Conditions Database, a 1:24,000-scale map-based system consisting of project location, rock, soil, and landform overlays. Data collected from Level 2 and 3 investigations are reduced to Level 1-scale accuracy and plotted on the database maps for use on future projects. The form of this database is planned to evolve from the current manual system into an integral part of a computerized Geographic Information System over the next several years.
Investigative Standards

Since the inception of the full-time practice of engineering geology in the Olympic National Forest, all project work has followed a set of standards. Rock and soil units designations are based on stratigraphic relations and physical characteristics. Soil units are classified in the field using the Unified Soil Classification System (Casagrande, 1948; American Society for Testing and Materials [ASTM], 1987). Rock units are classified in the field using the Unified Rock Classification System (Williamson, 1984). Designation of rock and soil units is completed at a scale appropriate for the investigative level.

For Level 1 and 2 investigations, a topographic map at the pertinent scale is used for plotting of field data, and specific conditions and interpretations relative to a given location are recorded in a field notebook. Site-specific (Level 3) investigative work follows the Field-Developed Cross-Section Method (Williamson et al., 1981). This method involves the measurement to scale, in section, of the distribution of terrain features and related subsurface characteristics of rock, soil, and ground water interpreted by the engineering geologist. Field-developed cross-sections are constructed using the Brunton Compass, hand clinometer, and cloth tape.

Level 3 investigations are commonly tied to engineering site surveys for reference. Subsurface exploration is conducted on Level 3 investigations as needed to confirm interpreted subsurface relations, to obtain soil and rock samples, and to conduct tests at specified locations and depths. The payoff for using these standards is a consistent methodology used by the project engineering geologist; this is essential to providing consistently high quality and reliable services to the client.

For specific project applications (usually during Level 3), representative samples of soil and rock units are tested in the materials laboratory for engineering quality and strength parameters following the testing procedures set by ASTM (1987) and the American Association of State Highway and Transportation Officials (1986). Laboratory testing is completed to confirm field testing and to provide more accurate data on materials characteristics and strength.

Analytical Standards

The goal of slope stability analysis is assessment of risk of failure. Prior to 1984, calculation of factor of safety for these involved "hand cranking" of values using hand-held calculators, or even slide rules. Slope stability analyses were completed only for site-specific investigations of slope movement features. Evaluations of broad areas for risk of failure were usually done subjectively; infrequently, the infinite slope equation was used for area analyses. (Methods of slope stability analysis are outlined in Morgenstern and Sangrey, 1978.)

With our acquisition of programmable hand calculators and personal computers, the speed and sophis-
tication of analysis increased several orders of magnitude. Prellwitz (1985, 1988; Prellwitz et al., 1985) employed these new tools and refined the three-level method of completing project work. Using the infinite slope equation as part of an algorithm for Levels 1 and 2, Prellwitz wrote the programs Slope Stability Infinite Slope (SSIS) and Slope Stability Infinite Slope for Critical Height (SSISCH) for use on the Hewlett-Packard 41 (HP 41) hand-held programmable calculator. (Critical height refers to the maximum height of an excavated road cut slope that can be constructed without slope failure.) Prellwitz (1985) also developed the program Slope Stability Method of Slices (SSMOS) for the HP 41 for use in site-specific (Level 3) analyses; it uses three methods of analysis: Fellenius or Ordinary Method of Slices, Modified Bishop, and Simplified Janbu. Armed with these three programs and the HP 41, the engineering geologist could now finish a major portion of his/her analysis in the field utilizing a field-developed cross-section. Hammond et al. (1988) have taken this work a step further with their program Level 1 Stability Analysis (LISA) for IBM and other compatible personal computers. The major differences between Prellwitz's calculator-based programs and Hammond's LISA are that the latter uses probability statistics and as many as 1,000 iterations of the infinite slope equation to determine failure probabilities by area.

The Forest Service engineering geologist has transitioned through the adoption of these programs from use of a mostly subjective process for planning (Levels 1 and 2) and a somewhat cumbersome but sophisticated slope stability analyses for site-specific (Level 3) projects, to use of easily applied, highly sophisticated computer programs for slope stability analyses for all project phases, from planning (Levels 1 and 2) through design and construction (Level 3). This has significantly increased project and program efficiency and effectiveness.

Analyses of rock and aggregate sources are also three-tiered. During Level 1, the quality of each designated rock or soil unit is evaluated to determine potential suitability for use, with the end-product being identification of new prospects to be investigated. During Level 2, rock and soil units are designated and evaluated for each proposed site. Suitability of each unit is evaluated, on the basis of the results of field tests, for the proposed use. The suitability and estimated quantity of rock and aggregate materials must be evaluated along with terrain characteristics and local environmental conditions and the distance of the site from the proposed application to determine whether any further investigative work is warranted. Level 3 analyses consist of synthesis of data developed during measurement of field-developed cross-sections, results of lab testing, and confirmation drilling; separation of materials into suitability use zones; and, where appropriate, slope stability analyses using the previously Level 3 methods for soils, plus Hoek and Bray's methods (1981) for analyzing wedge and planar rock failures.

To demonstrate how engineering geologists assigned to the Olympic National Forest completed project work before and after the advent of the "computer age", two case histories are discussed below.

**CHESTERSHIRE AND BACKDROP TIMBER SALE CASE HISTORIES**

Located within the Quinault Ranger District of the Olympic National Forest, the Chestershire and Backdrop Timber Sale planning areas overlap each other along Quinault Ridge and the upper reach of Chester Creek (Figure 3). Both are discussed here because they give a historical perspective of how a Level 1 project was completed in the late 1970s (Chestershire) and the mid- to late 1980s (Backdrop). Because of the overlap, data used for both areas are the same or similar.

**Level 1**

**Chestershire Timber Sale Planning Area**

An investigation of the Chestershire Timber Sale planning area was requested by the Quinault Ranger District in 1978. The stated purposes for the investigation were to:

1. Determine the probability of reactivating the Chester Creek landslide by reopening a segment of Forest Service Road 2261 that crosses the toe of the slide;
2. Locate corridors that would be suitable road location alternatives to the existing road crossing the toe of the slide; and
3. Determine effects of road development and timber harvest on Chester Creek and other potential slide areas.

The District Ranger was concerned that a potential increase in sedimentation into Chester Creek would occur as a result of harvesting timber and constructing roads in this planning area. Since Chester Creek was identified as an anadromous fishery, any increase in sedimentation would have a potential for causing both short- and long-term adverse effects. Because of these concerns, the District Ranger requested two investigations, one (Level 1) of the Chestershire planning area, and a second (Level 3) of the Chester Creek landslide. Two teams of engineering geologists were mobilized to conduct both investigations concurrently.

Aerial photographic interpretation and field mapping of the Chestershire Planning Area were completed in 1978. Field mapping was completed according to standards discussed in the introduction of this paper. Soil and rock units were designated and classified. Areas of saturated soils and slope movement features in soil and rock were identified and mapped. Slope percent was measured in the field or derived from existing
reactivating the Chester Creek landslide by area was made concurrently with the Level 1 study of alternatives and an evaluation of alternative rock conditions were specified for Level 3 (site-specific investigations) outcomes in each slope sources in the Chestershire planning area. using standards established in the introduction. Interpretations of subsurface data sets, which allowed the identification of critical factors related to slope stability and erosion. These critical factors are:

- Slopes greater than 70 percent;
- Hard rock units covered by shallow soil;
- Adversely oriented mass planar features in rock; and
- Saturated soils.

A slope stability/erosion zoning map was then compiled from these data (Figure 4). Zone 1 has stable slopes; Zone 2 has potentially unstable rock slopes; Zone 3 has slopes that have a low to moderate probability of slope movement; Zone 4 has stable slopes but a high potential for soil erosion; and Zone 5 has unstable soil slopes. Subjective estimates were then made of the potential of initiating slope movements from road construction and timber harvest activities; the scope of predicted effects was also estimated.

A short report (Neal, 1979) outlining predicted impacts in each slope stability/erosion zone was submitted to the Quinault Ranger District interdisciplinary timber sale planning team (ID team) as a reference for the timber sale environmental analysis report (EAR) as required under the NFMA. This report included a geotechnical assessment of transportation and logging alternatives and an evaluation of alternative rock sources in the area.

To answer the question regarding the probability of reactivating the Chester Creek landslide by reconstruction of the existing road, a Level 3 study of the slide area was made concurrently with the Level 1 study of the Chestershire planning area, using standards specified for Level 3 (site-specific investigations) outlined in the introduction. Interpretations of subsurface conditions were confirmed by drill exploration. On the basis of analysis of the slide data, Savage and Neal (1979) recommended that no timber harvesting, road reconstruction, or new road construction be conducted adjacent to or within the landslide area. Results of this investigation were also utilized in the Chestershire Sale EAR.

It should be noted here that while the the level of investigation conducted on the Chester Creek landslide was appropriate for the scope and nature of the problem, it is unusual for Level 3 standards to be applied to project planning and environmental assessment. This example shows, however, that the three-tiered system is a flexible standard that can fit most circumstances.

Backdrop Timber Sale Planning Area

Since the Chestershire planning area overlaps the Backdrop Timber Sale planning area (Figure 2), pre-existing data collected for the former were utilized for the latter as much as possible. Additional fieldwork to cover areas not previously mapped was completed during the 1986 field season. The objectives for this project were the same as those stated for Chestershire.

A series of overlay maps was compiled to augment those used for the Chestershire area. Included as a new overlay was a map of all proposed timber sale clearcut units (Figure 5). Each unit was analyzed to determine the probability of slope movement using the LISA computer program. The following data were used for these analyses: physical characteristics and depth of soil, slope percent, the ratio of ground-water depth to soil depth (dw/d), root shear strength, and tree surcharge.

As noted in the introduction, LISA analyses are completed using probability statistics; LISA generates as many as 1,000 iterations of the infinite slope equation. By using LISA, the engineering geologist provides more statistically meaningful results than those provided by the older subjective methods. Table 1 shows the probability of the occurrence of slope movements for each proposed timber sale clearcut unit under existing natural conditions (base level), after a 50 percent partial cut, and after clearcutting. From the data developed, it is apparent that many of the units are potentially unstable; slope movements are predicted to occur in some even without logging. Aerial photographic interpretation and field reconnaissance of existing clearcuts in the area indicate that the locations of slope movement features appear to correlate well with predictions generated with LISA. Although there are no large slope movement features in the area other than the Chester Creek landslide, there are a number of small features within each timber sale unit.

Calculations of safety factor for the Chester Creek landslide (Savage, 1979) indicated a high probability of failure. Koler (1988) synthesized the Chester Creek landslide data into a format suitable for LISA by using SSIS to back-calculate the physical parameters for input. Results from the LISA analysis for the Chester Creek landslide showed that under natural conditions 68 percent of the area would continue to fail without any timber harvest, as compared to 72 percent after a 50 percent partial timber cut, and 77 percent after clearcutting. These analyses provided a statistical probability for potential management alternatives that clearly shows that the earlier "no go" decision was valid.

Results of these analyses were compiled and documented (Koler, 1988) in a report for use in timber sale area environmental analysis.
Table 1. Level 1 Stability Analysis for Backdrop Timber Sale Planning Area; see Figure 5.

<table>
<thead>
<tr>
<th>Timber sale unit</th>
<th>Natural conditions</th>
<th>50%</th>
<th>Partial cut</th>
<th>Clear cut</th>
</tr>
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<tbody>
<tr>
<td>A</td>
<td>16%</td>
<td>26%</td>
<td>35%</td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>53%</td>
<td>70%</td>
<td>78%</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>32%</td>
<td>49%</td>
<td>60%</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>18%</td>
<td>37%</td>
<td>46%</td>
<td></td>
</tr>
<tr>
<td>D1/D2</td>
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<td>72%</td>
<td>77%</td>
<td></td>
</tr>
<tr>
<td>E1</td>
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<td>E2</td>
<td>26%</td>
<td>45%</td>
<td>61%</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>36%</td>
<td>49%</td>
<td>61%</td>
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<tr>
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<td>H</td>
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<td>55%</td>
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<td>58%</td>
<td>72%</td>
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<td>T</td>
<td>17%</td>
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<td>35%</td>
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</tr>
<tr>
<td>J</td>
<td>71%</td>
<td>86%</td>
<td>93%</td>
<td></td>
</tr>
</tbody>
</table>

Level 2

Chestershire Timber Sale Planning Area

The Quinault District Ranger decided, on the basis of reports by Neal (1979) and Savage and Neal (1979), to abandon the segment of Road 2261 that crossed the toe of the Chester Creek landslide. In 1981, the District Transportation Planner proposed two corridors for possible road construction (Figure 6). The first corridor was planned to provide access to slopes west of the Chester Creek landslide, upslope from Road 2261. The second corridor involved the extension of Forest Service Road 2220-011 along a broad glaciofluvial terrace above the west bank of Chester Creek. This extension was planned to provide access to slopes west of Chester Creek and to tie back into Road 2261 southwest of the Chester Creek landslide. A third corridor would provide temporary access across the middle part of the Chester Creek landslide to access timber northeast of the slide, and a fourth was planned (after the third was dropped) to access that same timber from the east. Level 2 geotechnical work along these corridors consisted of interpretation of aerial photographs and reconnaissance mapping along proposed alignments at 1 in. = 300 ft (1:3,600) to substantiate Level 1 interpretations and to supplement the detail of available data. During this process, each corridor was divided into terrain segments, and typical ground conditions were sketched in cross-section. Data and information were synthesized and evaluated, and several short reports were written (McBane, 1981a, 1981b; Neal, 1982a, 1982b). The Quinault District Ranger's decisions, formed on the basis of these reports, included constructing a road along the second corridor, dropping the first and third corridors, and, tentatively, planning to construct a road along the fourth corridor at a later time.

In December 1983, Neal conducted a Level 2 investigation of an area at the toe of the Chester Creek landslide to evaluate the potential for increased slide activity that might result from logging there. As a result of his report (Neal, 1983a), a follow-up memo (Neal, 1983b) and subsequent meetings with District and Supervisor's Office staffs, the Chester Creek landslide was removed from further consideration for timber harvesting.

As with the Level 1 analyses, this Level 2 assessment predated Prellwitz's computer programs. Field and office analyses at that time were still "hand cranked". Since many of the recommendations were based on subjective conclusions, considerably more communications were necessary to reach the recommended decision than have been necessary since probability analyses using LISA have been completed on similar slopes.

Backdrop Timber Sale Planning Area

When the Backdrop area was first designated for timber sale planning, a great deal of attention was focused on the Quinault Ridge Road (Forest Service Road 2258, Figure 5). Prior to any planning of timber sale units or boundaries, the Quinault District Ranger requested a geotechnical report for transportation planning along this road because it is the only access along Quinault Ridge and numerous road-associated slope movements have closed it. A geotechnical reconnaissance for reconstruction of the road was conducted by McBane (1981). As of early 1988, no transportation planning for the Backdrop Timber Sale has been finalized. To complete transportation planning, the 1981 geotechnical work by McBane would need updating because of more recent slope movements. Future slope stability analyses will be conducted using Prellwitz's SSISCH and rock slope analysis methods developed by Hoek and Bray (1981).
Appendix 1.1

Level 3

Chestershire Timber Sale - Extension of Forest Service Road 2220-011

A site-specific geotechnical investigation was conducted in 1982 along the preliminary survey line (P-line) for design of the 2220-011 tie road along Corridor 2 (Figure 5). This investigation built upon data collected during Levels 1 and 2 project work. The project engineering geologist divided the P-line into terrain segments. Soil, rock, topographic, and ground- and surface-water conditions were used as criteria for delineating segments. The purpose of segmentation was to provide data and portray conditions that represented typical terrain characteristics, and, in turn, to enable the development engineer to select a design that was appropriate for ground conditions encountered for each segment. The project engineering geologist completed slope stability analyses for areas of existing slope movement features, or which appeared to have the potential for slope movement to be initiated by road construction. In addition to small slope movement features, conditions encountered included steep slopes with saturated soils and spring lines, plastic soils with low shear strength values, and rock slopes with inherent adverse discontinuities.

The Modified Bishop Method of Slices (Bishop, 1955) was used to analyze features with ongoing or predicted rotational movement. The infinite slope equation was used to analyze features with ongoing or predicted translational movement. Different design alternatives were evaluated for conditions in each segment. The project engineering geologist presented analyses of alternatives and recommendations in his report (Koler, 1982).

The Wineglass pit, located roughly 6 mi northeast of the project, was selected for production of pit-run aggregate. This pit had been investigated previously; the last phase of drill exploration there prior to the Chestershire road construction was completed in 1981. The Geotechnical Section provided a rock source operating plan to be included in the timber sale contract package (Blair, 1982).

The extension of Forest Service Road 2220-011 has been completed. The construction phase involved considerable consultation with an engineering geologist, primarily dealing with the suitability of subgrade materials for use at constructed grade level (Neal, 1985; Jordan, 1988). To date, no new logging has been conducted in the area because of decreased demand. Resale of some of the old Chestershire timber sale units is contemplated.

Backdrop Timber Sale - Road 2258000, Station 67+76 to 67+90

In 1983, subsequent to McBane’s Level 2 investigation of Road 2258000 for transportation planning, a site-specific (Level 3) investigation was conducted of the segment of Road 2258000 between Stations 67+76 and 67+90 to evaluate alternative ways to regain adequate width for log haul. Road 2258000 was needed for log haul for the Binder timber sale area, located on the west slopes of Quinault Ridge. A team comprised of engineering geologists and a geotechnical engineer investigated the site using the field developed cross-section method, Unified Rock Classification System, and Unified Soil Classification System, as outlined in the introduction. Subsurface conditions were confirmed and foundation strength characteristics were measured during drill exploration (refer to Figure 7, Geotechnical Cross Section 30 for Binder Timber Sale). Road alignment alternatives were analyzed using a yarder simulator. Construction of a Hilfiker retaining wall to support the roadway was selected as the preferred alternative. Design analyses were completed using the Region 6 retaining wall design guide (Driscoll, 1979). Conclusions and recommendations were documented in a report written for design (Blair and Scheible, 1983).

Binder timber sale was dropped from the program because of environmental and economic considerations, so the retaining wall was not constructed. However, with upcoming planned logging of the Backdrop timber sale area, a contract was prepared in 1987 for wall construction in 1988. After award of the contract (and after the snow had melted), additional slope movement, which resulted in loss of all remaining soil in the foundation area, was discovered. The project engineering geologist remeasured site conditions and is currently working on modifying the wall design (Jordan, 1988).

CONCLUSIONS

Employment of engineering geologists in the U.S. Forest Service increased significantly in the mid-1970s after enactment of federal environmental legislation. The Olympic National Forest began employing engineering geologists as part of its geotechnical staff in 1974. Engineering geologic services have greatly expanded from those early years, from investigation of rock and aggregate sources and road corridors to investigations for area planning, water well and drainfield location and design, and, most recently, review of small hydroelectric project applications. Since the mid-1970s, the geotechnical staff employed by the Olympic National Forest have been assigned to the Supervisor’s Office, with their duty station at Ft. Lewis, Washington. The Geotechnical Section is comprised of a team of engineering geologists, geotechnical engineers, civil engineering and lab technicians, and a drill crew. While each of these fields is essential to the production of complete geotechnical services, none can operate independently of the others. Having all skills in a centralized location fosters teamwork.

Engineering geologists on the Olympic National Forest staff complete work within a scale-dependent, three-tiered system. Level 1, the lowest tier, comprises
the area and project planning stage; data are collected and mapped to a minimum 1:24,000 scale. Level 2 involves reconnaissance mapping of corridors for transportation planning or road or facilities location, at a scale of 1:3,600. Level 3 is site-specific, and mapping is completed to a scale of 1:600 or greater. This paper traces the evolution of this three-tiered method of work over the past 10 yr by examining two overlapping projects, the Chestershire and Backdrop Timber Sale areas, located on the Quinault Ranger District of the Olympic National Forest. During this period, the "computer age" has come into being for engineering geologists in the Forest Service. The Chestershire Timber Sale examples demonstrate past methods, which included subjective evaluations of broad areas and analyses of mathematical models developed for specific sites. The Backdrop Timber Sale examples demonstrate current methods, with expanded use of mathematical models to predict probabilities of slope movements over broad areas, with assistance from programmable calculators and personal computers. The net effect of this computerized analysis is a statistical probability of slope movement for each proposed activity during each project phase, from planning through design and construction. This adds credibility to geotechnical conclusions and recommendations and facilitates more objective management decisions regarding resource management and use.

REFERENCES


SECTION 2

INITIAL SLOPE STABILITY ASSESSMENT
IN RESOURCE PLANNING

Principal contributors:

Ken Baldwin, Engineering Geologist
(Section Leader)
USDA Forest Service
Klamath National Forest
Happy Camp, CA 96039

Courtney Cloyd, Forest Engineering Geologist
USDA Forest Service
Siuslaw National Forest
P.O. Box 1148
Corvallis, OR 97339

Juan de la Fuente, Forest Geologist
USDA Forest Service
Klamath National Forest
Supervisor's Office
1312 Fairlane Road
Yreka, CA 96097

Tom Koler, Engineering Geologist
USDA Forest Service
Intermountain Research Station
1221 S. Main
Moscow, ID 83843

Mark Leverton, Engineering Geologist
USDA Forest Service
Willamette National Forest
South Zone Engineering
49098 Salmon Creek Road
Oakridge, OR 97463

Michael T. Long, Forest Engineering Geologist
USDA Forest Service
Willamette National Forest
P.O. Box 10607
Eugene, OR 97440

Thomas K. Reilly, District Ranger
USDA Forest Service
Walla Walla Ranger District
Umatilla National Forest
Walla Walla, WA 99362

Cindy Ricks, Resource Geologist
USDA Forest Service
Siskiyou National Forest
Westside Engineering Zone
93976 Ocean Way
Gold Beach, OR 97444
# Section 2. Initial Slope Stability Assessment in Resource Planning

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2A. Introduction

Kenneth Baldwin, Engineering Geologist, Klamath National Forest
Juan de la Fuente, Forest Geologist, Klamath National Forest

Figure 2A.1.—Slope stability analysis flowchart. Section 2 concerns the four shaded sections shown.

2A.1 Purpose and Organization

Section 2 outlines concepts useful in characterizing landslide processes at a river basin scale.

The behavior of landslides and associated processes—such as creep, raveling, piping, and surface erosion—varies considerably both in space and time. The gradients along which soil and rock materials move are influenced by tectonic uplift, river incision, volcanism, and glacial activity. Knowledge about these slope-forming processes is useful in:

(1) Supplementing understanding of site-scale landslide behavior;
(2) Assessing cumulative effects;
(3) Designing a local program for management of the effects of landslides; and
(4) Defining ecological units.
Investigations of these processes and their products are used, for planning purposes, in descriptions of the causes and effects of landsliding.

Evidence of past occurrences of landslides can be used to forecast future behavior of the landscape. Understanding the climate, geologic and geomorphic history, and seismicity of the area is essential. Local patterns in the character of landslide processes provide clues to site processes that are useful in the design of remedial measures and preventive management practices. Landslides and associated processes influence the distribution and structure of ecological habitats, particularly riparian habitats. Opportunities to integrate a range of investigations into descriptions of landscape behavior become apparent as this investigation matures.

The technology necessary to manage slope stability, particularly at the scale of river basins, continues to grow. Landslide damage to hillside homes in Los Angeles in the winter of 1951–1952 led to adoption of the first city grading ordinance in the United States (Scullin, 1983). By 1965, most large cities in the western United States had adopted similar ordinances. Improvements to the grading ordinances have been prompted by local experience with damaging effects of landslides and the erosion associated with subsequent storms. The technology of landslide and erosion control in urban areas is still evolving. Much effort has been spent recently developing ordinances addressing seismic safety.

Future developments in technology are anticipated. Earth-orbital sensors are sending continuous streams of data that can be interpreted for surface hydrology, geomorphology, geology, and geodesy. The computing hardware necessary to analyze these data is affordable and will fit on your desk. In the near future, we can expect substantial improvements in the speed and volume of data processing. At the same time, analytical methods are being obscured in computer software. Assessment methods developed today need to be designed to accommodate new data and for translation to future data processing systems.

There is still much to be learned about potentially catastrophic processes such as landslides, floods, and earthquakes. Occurrence of such events is bound to provide a rich variety of new data and an opportunity to improve the quality of assessments substantially.

The scientific method provides a systematic approach to the assessment of landsliding. Knowledge about the mechanics and distribution of landslides increases with observation and interpretation of features and processes on the landscape. Interpretations are tested on similar ground in another area or at another time. Documented observations and conclusions reviewed by peers are added to the knowledge base. With increased knowledge and experience, the efficiency and economy of landslide management is improved. (See section 3A for a discussion of problem definitions and applications, and appendix 3.1 for a complete discussion of the scientific method.)

One of the basic laws of interpreting geology is that of uniformitarianism. It has commonly been restated as "the present is the key to the past." The assessment of erosion and landslide processes adds a new twist to the concept: "the past is the key to the future." In the classical sense, uniformitarianism means that in interpretation
of the rocks, we assume the processes responsible for their character could or do occur in modern times. In landslide assessments, we assume the processes interpreted to have been acting in the past will occur again in the future.

The foundation of description and analysis is the model created to represent the behavior of typically very complex geomorphic systems. Measurable components of the real system are represented quantitatively. If measurements are not available, the value of the components is estimated or assumed. In developing a model, the focus is on those elements with strong influences on the system and identified values; minor influences may be generalized or omitted from the model. Models are often constructed of arbitrarily defined elements (stratigraphic units, landslides, etc.). The number of defined elements may be optimized by iteration of calculations, using varying numbers of elements, to identify the minimum number that provides maximum useful resolution. If assumed or approximated values have a large influence on the value of results, then a large number of elements may provide a false sense of improved resolution. The results of models composed of fewer elements are the easiest to comprehend.

Model elements should be designed and named so that the user is presented with a useful picture of the behavior of the landscape. Descriptive terminology must be clearly defined, and typically a reference to the source of definitions is provided to the user. For example: *Debris avalanches (as defined by Varnes, 1978) derived from graywacke (as defined by Hutton, 1795) occur most frequently in the inner gorge (as defined by Kelsey, 1987).* Some terms have been used to describe such an assortment of objects that they may not convey a useful idea to the user.

2A.4 Design of Investigations

2A.4.1 Objectives

Project objectives are best developed interactively with the user (typically a land manager). The project design is controlled by the availability of data, technology, budget, and time. Varnes (1984) reminded us that funding, available personnel, and time constraints will inevitably dictate what can be done, but we should not allow these factors to determine at the outset what should be done. From the beginning, it is important to have an intuitive understanding of the nature of the conclusions. Relatively simple questions that can be answered by clean analytical procedures and clear documentation are likely to provide useful conclusions.

2A.4.2 Technical and Administrative Influences

It is the geologist's responsibility to determine the technical feasibility of attaining project objectives. Documentation of other similar investigations may help to assess feasibility; the lack thereof may result in critical difficulties. In such cases, it may be useful to answer a portion of the question, which may lead, in combination with the work of others, to additional future investigations of larger questions. Building scientific knowledge is analogous to standing on the shoulders of others, and in turn carrying others on your shoulders. Figure 2A.2 outlines elements common to many investigations of landslide processes.
It may be necessary to analyze a sample of the population when the population of objects under investigation is large. It is important to keep the design simple enough that users of the results are able to understand the conclusions; therefore, the design of statistical analyses will derive substantial benefit from a review by a statistician for simplicity and validity.

Multiyear projects are difficult to complete in an environment of annual financing. Look for ways to secure future financing or to accomplish the investigation in one year. This is very important in landslide investigations involving monitoring.

2A.4.3 Risk and Hazard

The terms “risk” and “hazard” suffer from widespread confusion about their meaning. Make sure that the user understands the meaning of these and similar terms. When assessing the probability of a landslide occurring, you must specify the area and time period over which the assessment is made. At a specific site, the probability of a landslide occurring today might be very low. At the same site, the probability of occurrence of a landslide during the next 10,000 years may be very high. This problem is resolved by defining the timeframe of the assessment and achieving a good understanding of the processes where landslides have occurred in the past. The list of resource values is site-specific, and landslides affect, among other things, water quality, fish habitat, soil productivity, utility of roads, reservoirs, private property, and human safety.

2A.4.4 Resolution and Suitability of Methods of Assessment

Many assessments of landslide hazards are done with computer models that try to include all of the relevant landscape features and processes (such as landforms, climatic influences, meteorological influences, soil properties, slope-forming processes, and hydrologic processes). Microscale processes have macroscale effects that are not well understood and, in many cases, are treated inaccurately by models. Very often, model parameters are indices rather than measures of actual behavior of processes.

It is critical to consider the relationship between the design of a specific investigation and the utility of available analytical methods. The rapid expansion of technology provides us with instruments, software, and models involving complex processes developed for specific applications. When the specialist uses model designs, analytical procedures, and instruments without fully understanding how they work, and without adjusting for local conditions, he or she may reach critically defective conclusions.

Models vary in their ability to produce realistic results. For example, an investigator dispatched to a distant location in response to an episode of catastrophic landsliding might arrive with good academic preparation and experience from other areas but with little knowledge about the local patterns in geology, geomorphology, hydrology, climate, and seismicity. The assessment made from a vehicle tour of the area would likely be very different from that made after a year of local practice. As we come to understand the activity of specific, local processes, our model can be modified to improve resolution. Existing analytical models used in landslide management occupy
Figure 2A.2.—Elements common to many landslide process investigations.
a wide range in the spectrum of resolution. It is important to understand where each model lies in the spectrum and to be prepared for evolution of the methods and data quality and quantity.

It is also important to indicate the quality and applicability of conclusions. For example, one method of assessing future landslide hazards may provide a good indication of the volume of landsliding expected from a certain precipitation event across a large area but not indicate source sites. Another method may very well indicate sites of future landsliding but not provide an estimate of the cumulative landslide production potential.

2A.4.5 Local Values and Conditions

Project objectives and localized landscape processes should be accommodated when using regional or national investigation techniques as a template for investigations. For example, an assessment of reservoir sedimentation may be generally similar to an assessment of effects on fish habitat but differ in the timing of processes. Other discrepancies may arise as a result of differences in climate, geology, topography, seismicity, and human values.

2A.4.6 Scale of Investigation

The map scale of an investigation should be selected in conjunction with the objectives. The map scale must be large enough to allow the smallest of the significant features to be mapped. The range of choice may be constrained by the scale of existing maps which may be enlarged, but the accuracy of topographic information will remain that of the original scale. Custom topographic maps can be made at reasonable cost from aerial photography. The accuracy of large scale (1:30,000 or larger) maps may be limited in forested environments where contours are drawn on the forest canopy. Where the depth of the vegetation is irregular, typically greater along water courses, the topographic model will be smoother than the actual ground surface. A good grasp of cartography and photogrammetry is needed to do high resolution, quantitative analysis of map information. If the map scale is too large, information may be classified into many units of small size, and the resulting map may not provide a useful image.
2B. Climatic, Geologic, and Hydrologic Influences on Slope Stability

Kenneth Baldwin, Engineering Geologist, Klamath National Forest
Juan de la Fuente, Forest Geologist, Klamath National Forest
Cindy Ricks, Resource Geologist, Siskiyou National Forest

2B.1  Delineation of Geomorphic Terranes

Geomorphic terranes are units of the landscape defined by similar soil and rock properties, slope-forming processes, topography, climate, and vegetation. Landscapes of western North America have relatively complex geomorphic histories that result in complex landscapes. To make analysis easier, landscapes can be divided into units. The behaviors of landslides and other hillslope and channel processes occurring in distinct geomorphic terranes often require unique analyses. For instance, debris slides in steep terrain of shallow soils respond to high-intensity precipitation occurring over time scales of hours to weeks, while earthflows respond to precipitation over time scales of weeks to years. When associated with an historical landslide inventory, geomorphic terranes can be used to estimate landslide sediment production and delivery over large areas. The Salmon River Basin sediment analysis, presented in section 2F, is an example of this approach.

The site conditions for geomorphic terranes that influence landsliding are either intrinsic characteristics of the site (such as soil mechanics properties, soil hydrologic properties, slope gradient, and vegetation composition and distribution) or extrinsic characteristics (such as climate, precipitation intensity, and seismic loading). It is these extrinsic influences that often trigger landslides. By analyzing the landslide response to past events, we can estimate the threshold intensity and frequency of landslide movement. The spectrum of types of landslide episodes is bounded by combinations of extremes of site conditions.

Landslides may be classified using such criteria as mechanism and velocity of movement, character of the soil and rock, mode of deformation, geometry of the moving mass, and moisture content. This variety of classification schemes presents difficulty, and the landslide classification scheme most commonly used is that of Varnes (1978). Varnes distinguished two types of premovement materials: bedrock and soil. Soil is described by the dominant grain size and by moisture content. Rock masses are described by the frequency or lack of discontinuities, their orientations, and shear strength. Landslides are further described by rate of movement. It is important to identify the classification scheme in use or to provide definition of these terms in these observations.

2B.2  Precipitation

Climate has a strong influence on the distribution of water in and on the landscape. It is important to have a good characterization of the range of precipitation rates in both site stabilization design and assessment of effects. Landsliding may be initiated
by precipitation intensity and duration on a scale of inches per hour to inches per decade.

Consider the variation of landslide response between an intense thunder shower on an otherwise dry landscape and a season of continuous, light rainfall on a landscape saturated by antecedent rain. In the intense thunder shower, the intensity of precipitation interplays primarily with soil permeability and the efficiency of surface drainage. In this case, we might expect to see mass movement dominated by surface-erosion and debris-flow processes. In a season of prolonged light rainfall, the permeability of the landscape and efficiency of subsurface drainage may be the limiting factors, and mass movement may be dominated by subsurface erosion (such as piping) and movement of earthflow landslides. These two situations might be viewed as end members of a spectrum of precipitation that contributes to landsliding.

A given landscape will respond uniquely according to the character of other elements of landsliding as well as rainfall. Where the rates of change associated with other elements (such as soil formation, soil textural evolution, and landform evolution) are relatively very slow, characterizing precipitation associated with past landslide episodes is the best way of assessing future behavior.

2B.2.1 Short-Term Precipitation

Widespread debris-flow activity that occurred on windward hillslopes from northern California to Utah in January 1982 and February 1986 is a good example of landslide response to intense short-term precipitation. In these cases, landsliding occurred in the middle of the winter rainy season, so the influence of antecedent precipitation presumably was important. What was learned has been summarized by Keefer et al. (1987). Previous investigations (Keefer et al., 1987) reported hourly precipitation rates and antecedent rainfall (expressed as amount since beginning of the rainy season). This opens the possibility that the return probability of the precipitation landslide event might be calculated. Keefer et al. (1987) suggested that an empirical formula can be developed for local conditions using estimates of the rate of loss of soil water and the critical volume of water in the soil at the time of landsliding. Similar assessments of debris-flow activity can be made for summer thunderstorms.

Case History: Short-Term Precipitation Intensity as a Trigger of Debris Sliding

In November 1977, 4 days of continuous and intense rainfall resulted in widespread debris avalanching in the southern Appalachian Mountains. Neary and Swift (1987) described the methods and results of precipitation monitoring of the event. They found 4 days of continuous precipitation at 20–50 mm/day, peak precipitation rates at 21–102 mm/hr, and 177 percent of normal rainfall during the 2 months prior to the event. Progress of the storm was monitored by means of GOES infrared satellite imagery.

By the methods used, real-time forecasting of debris-flow hazards was possible. In addition, threshold conditions for the event of widespread debris avalanching were recognized. Although the calculated return probability of such storms only exceeds one in 50 years, these storms play an important role among geomorphic processes in the region.
Similar studies have been conducted by Campbell (1975), Cannon and Ellen (1985), Hollingsworth and Kovacs (1981), Reneau et al. (1984), and Wieczorek (1987).

2B.2.2 Long-Term Precipitation

Where annual precipitation is relatively high, variation occurring over decades may have a significant influence on the occurrence of landslides. On the Pacific coast of North America there is typically a strong decreasing gradient in annual precipitation from the coast to the east side of the coastal mountain ranges. As a result, a broad spectrum of landslide regimes is present.

In northern California, the annual precipitation gradient is about 1 inch of rain per mile from the coast. Near the coast, average annual precipitation approaches 100 inches. At Yreka, on the north flank of Mount Shasta about 100 miles from the coast, average annual precipitation is about 18 inches, ranging from 6 to 33 inches. At Happy Camp, on the Klamath River 30 miles from the coast, average precipitation is about 50 inches, ranging from 23 to 88 inches. Much of the difference in landslide behavior between these two locations can be explained by analysis of the weather records (see the case history below). Most interesting is the effect on landslide behavior of the larger range of variation of annual precipitation at Happy Camp. The distribution of average annual precipitation through time is not random. Years of below-average precipitation tend to be associated in time. The result is occasional, prolonged climatic wet periods, with which is associated most of the historical landslide activity. This effect may not be so pronounced where the long-term range of annual precipitation is smaller.

Case History: Distribution of Average Annual Precipitation and Landsliding in the Central Klamath Mountains, CA

A detailed picture of the local climate is useful in understanding the behavior of erosion, landsliding, and the structure of stream channels. The annual precipitation during the period of record (1915–1990) at Happy Camp ranges from 23 to 88 inches. The average annual precipitation at the ranger station is about 51 inches. Annual precipitation increases with elevation and to the west is 80 or more inches per year. About 90 percent of precipitation occurs from October through May during north Pacific cyclonic storms. The remainder occurs during summer thunderstorms. The snow level varies from valley floor to summit elevation during the winter season. Winter precipitation occurs mainly as snow above 3,000 feet in elevation, and mainly as rain below that elevation. The fluctuation of snow level occasionally results in rain falling on snow, causing it to melt rapidly.

The precipitation record is characterized by two distinct climate trends (Baldwin and de la Fuente, 1986; Coghlan, 1984). These alternating trends of wet and dry conditions last for a few decades. Climate change is not understood well enough to forecast shifts in trends. Approximately 40 inches average annual precipitation falls in the drier trends, and about 56 inches falls during the wetter trends (see table 2B.1 and figure 2B.1). In comparing these precipitation trends with the snow depth record from Crater Lake, OR,
It is apparent that fluctuations in temperature influence the proportion of precipitation falling as snow. The Happy Camp record, along with longer records from Eureka and other stations in northwestern California, indicate that the period from 1870 to 1910 was a wet time, 1911 to 1937 dry, 1938 to 1975 wet, and 1976 to present dry. Similar fluctuations in climate have been described by Coghlan (1984) for northwestern California, by Meko and Stockton (1984) from stream flow records in the western United States, and by LaMarche (1974) from interpretation of tree rings in the White Mountains of California. Significant exceptions to the pattern include the over 80 inches of precipitation of the 1982 and 1983 seasons associated with the relatively strong El Niño/Southern Oscillation event and volcanic eruptions of that time.

Table 2B.1.—Temporal distribution of annual precipitation showing climatic trends, 1915–1990, Happy Camp, CA.

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<tr>
<td>80–90</td>
<td>6</td>
<td>0</td>
</tr>
<tr>
<td>90–100</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Average</td>
<td>50.7</td>
<td>40.6</td>
</tr>
</tbody>
</table>

* Range of Observations: 23.3–88.5 inches

In this climatic regime, the occurrence of landslides in geomorphic terranes of perennial, perched ground water is closely associated with periods of above-average annual rainfall. Activity of earthflows and attending debris-flows occurs during series of years with higher levels of surplus precipitation and residual ground water from preceding years of surplus precipitation. Because intense precipitation (on a scale of inches per hour) also occurs in years of above-average annual precipitation, episodes of widespread landsliding of thin colluvial soils may also occur. Episodes of widespread
Landsliding in thin soils occurred in 1964, 1972, and 1974, in addition to earthflow activity.

From 1982 to 1986 many earthflows were active, but the rate of debris sliding in thin soils on steep slopes was low. This behavior has two possible explanations: (1) Precipitation intensities necessary to activate many debris slides were not achieved during this period, or (2) The potential for debris sliding was reduced by the occurrence of debris slides during intense precipitation in the period 1964–1975 (or both). The true explanation might be answered by further analysis of the precipitation record.

Analysis of aerial photography taken since 1944 and the lack of episodic landslide evidence prior to this century has led to the conclusion that about two-thirds of the landslide production of this century occurred during or soon after the storm of 1964–1965. The bulk of the remaining third occurred in the 10 years after that storm event.

When the precipitation trigger has been characterized in terms of short- and long-term time distribution, the return probability of such an event can be approximated. In western North America, the greatest problem is the lack of a long-term precipitation record, the longest records being only about 100 years old. This results in poor quality estimates of events that have a return probability less than a few times in a hundred years. However, records longer than 100 years have been interpreted from tree-ring analyses. The dendrochronology laboratory at the University of Arizona is the source of much of this work. When using climatic interpretations of tree rings, it is important to remember that ring development is
influenced by the seasonal distribution of precipitation and temperature, as well as by annual precipitation. Helley and LaMarche (1973) used alluvial stratigraphy and radio carbon dating of included wood to recognize floods of the magnitude of the 1964–65 event during the last thousand years in the Klamath Mountains. Because evidence of some floods may have been obscured by subsequent events, this can be considered the minimum frequency. Pollen stratigraphy of lake deposits in western North America is being interpreted for climatic information relevant to Quaternary geomorphic history. This can be an important aspect of landslide assessments because many of the erosion processes occurring today are strongly influenced by landform evolution over the past hundred thousand to few million years.

Landscape features that originated in response to a past climate continue to influence present landscape behavior. When local interpretation of paleoclimate is lacking, the work of others in nearby comparable regions may be used. Reneau et al. (1986) noted a clustering of minimum ages of radio carbon-dated colluvial deposits of coastal northern California around 12,000 years ago, suggesting a time of higher rate of landsliding. Adam and West (1983) and Adam (1988) have described the transition from the last glacial advance that occurred 10,000 to 15,000 years ago. Their interpretation of pollen stratigraphy at Clear Lake (Lake County, CA) suggests that climatic warming and substantial drying occurred during the transition.

**Case History: Influence of Late Quaternary Landforms on Landslide Distribution in the Central Klamath Mountains**

In the central Klamath Mountains of California, a large proportion of landslides originates in thick, red residual soils that lie on broad terraces a few hundred feet above river elevation. Infrequent rounded quartz boulders found in the otherwise fine-grained soil suggest that these soils developed from alluvial deposits. Based on radio carbon dates of similar soils in the Pacific Northwest, Dietrich (personal communication, 1989) believes these soils are typically over 40,000 years old. These surfaces are interpreted as being formed during a period of climate wetter than that of Holocene time and possibly lower levels of tectonic activity (Baldwin and de la Fuente, 1987 and 1989). Soils developed of deeply weathered granitic rocks occupy concordant topographic positions. Rates of landslide production from these soils are two to six times higher than metamorphic terrane in the same area (de la Fuente and Hessig, 1992). The elevated landslide rate is attributed to Holocene uplift and the incision of river channels through the deposits, 50-inch annual rainfall, and mechanical properties of the soils. Under the influence of the modern climate, large (up to 10-acre) earthflow landslides are common in the weathered alluvial/colluvial deposits. Debris flow is the dominant landslide process in the granitic residuum. Topographic expression of old scarps of earthflows of a few hundred acres indicate more extensive activity of earthflows in the past, presumably under wetter climates and possibly associated with great earthquakes of the Cascadia subduction zone.
2B.3 Tectonics and Seismicity

Additional variation in expected landslide response is created when the seismic loading is considered. At this writing, seismic hazard assessment is in a state of rapid growth. New, active faults are being recognized, the mechanics of earthquakes is being widely studied, seismic design of structures is evolving rapidly, and earthquake response programs are being developed. The Cascadia subduction zone is located off the western coasts of California, Oregon, Washington, and British Columbia. Heaton and Hartzell (1987) described evidence of the potential for great earthquakes in the northern portion of the Cascadia subduction zone. Clarke and Carver (1992) described evidence of the potential for great earthquakes in the southern Cascadia subduction zone.

Case History: Seismically Induced Landslides Based on Paleoseismicity

A series of investigations of paleoseismicity at Puget Sound, WA, constitute an impressive interdisciplinary assessment of seismic hazards. Adams (1992) has summarized progress of investigations during the past few decades as a preface to a series of reports of site investigations. The case for seismic hazards at Puget Sound and elsewhere in the Cascadia subduction zone (Vancouver Island to northwestern California) is built on evidence of: (1) Rapid uplift in the stratigraphy of marine terraces and tidal flats (Bucknam et al., 1992), (2) Evidence of tsunami episodes in the stratigraphy of tidal marsh sediments (Atwater and Moore, 1992), (3) Earthquakes interpreted from lacustrine turbidites (Karlin and Abella, 1992), (4) Dating of earthquake-induced landslides from drowned forests caused by rock-avalanche deposits (Schuster et al., 1992), and (5) Seismically induced lacustrine landslide deposits (Jacoby et al., 1992).

Liquefaction under a seismic load depends on the presence of ground water in primarily, but not exclusively, cohesionless soils. A landscape that holds ground water all year long has higher exposure to seismically induced liquefaction hazards than does one that drains efficiently. The science and technology of soil liquefaction has been reviewed by the Committee on Earthquake Engineering of the Commission on Engineering and Technical Systems of the National Research Council (1985).

Estimation of earthquake size for seismic hazard assessments is a young field of investigation in which a large amount of literature has accumulated in recent years. DePolo and Slemmons (1990) described five approaches to earthquake size analysis: (1) historical earthquake pattern; (2) paleoseismic interpretation; (3) source characterization; (4) regional (indirect data) interpretation; and (5) comparison with similar tectonic domain. Typically, several approaches are used in a study in order to reduce uncertainty of the analysis. Analyses discriminate among four types of earthquakes: (1) characteristic earthquakes, those typical of a particular fault; (2) maximum earthquakes, the largest to occur in a given time period; (3) maximum credible, the largest reasonably, physically possible; and (4) floating earthquakes, those that occur from unidentified sources, such as along structures adjacent to a known fault zone (dePolo and Slemmons, 1990).
Variations in ground water retention due to soil properties may be related to a number of geologic and geomorphic conditions. Where bedrock is of igneous origin and of low topographic relief, the increased susceptibility to weathering of the rocks may result in thick, cohesive soils with a relatively large ground water storage capacity. In contrast, a landscape of low-grade metasedimentary rocks and high topographic relief may result in thin, coarse granular soils with less ground water retention. Higgins and Coates (1990), Higgins et al. (1988), and LaFleur (1984) have contributed to the knowledge of the role of ground water in geomorphic processes.

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**Case History: The Influence of Short-Period (Storm Associated) and Long-Period (Seasonal) Ground Water Pressure Waves on the Activity of an Earthflow**

Iverson and Major (1987) described the influence of ground water pressure waves associated with storm- and season-scale precipitation on the activity of an earthflow on a tributary of Redwood Creek in northwestern California. The Minor Creek Landslide was selected as being typical of earthflows that occur extensively in Franciscan metamorphic terrane in the Coast Ranges of northern California. Site precipitation was correlated with piezometer and extensiometer data collected from 1982 to 1985. Accelerated movements of the landslide occurred in close association with nearly vertical pore-pressure waves caused by rainstorms. Storm waves were attenuated before reaching the base of the landslide. Seasonal pressure waves associated with above-average annual precipitation controlled attainment of threshold piezometric levels.
2C. Basin-Scale Assessment of Geologic Hazards

Cindy Ricks, Resource Geologist, Siskiyou National Forest

2C.1 Relative Hazard Zonation

Hazard and risk zonation approaches vary in their degree of resolution (accuracy), reproducibility (precision), and applicability to other areas, as well as the type of information provided. The “empirical” approach is based on local experience and emphasizes identification of unusual ground conditions. Typically, the probability of failure is not quantified, and potential failure sites may not be located unless a field-intensive approach is used.

Soil mechanics approaches to hazard zonation rely on stability analysis using universally applicable deterministic or probabilistic equations. Probability and resolution are quantified, and feedback from site investigations may be incorporated into a data base. The effects of variable harvest prescriptions can be estimated without local experience. This approach may require supplemental detailed site investigations to locate potential failures more accurately than for the above methods. A generalized and cross-referenced basin study checklist is located in appendix 2.1.

2C.1.1 Empirical Approach

Hazard delineations are typically based on lithology, structure, slope, geomorphic features, and landslide distribution. Consequences are commonly included to produce a risk assessment map. Varnes (1984) provided a comprehensive review of such approaches around the world.

Hicks and Smith (1981) described field-based hazard mapping using geomorphic and vegetative indicators of instability. This mapping includes an evaluation of past activity on the site.

2C.1.2 Statistical Approach

Statistical approaches, such as linear discriminant analysis, provide estimatable precision and accuracy. The variables are surrogates for processes and must be handled as additive or multiplicative factors.

Rice et al. (1985) have developed linear discriminant functions to distinguish stable from unstable sites. Additional testing of these functions (Neely and Rice, 1990; Lewis and Rice, 1990) indicates the importance of geomorphic and soil and rock variables, which require field measurement.

2C.1.3 Soil Mechanics Approach

The LISA computer program (Level I Stability Analysis Ver. 2.0 1991) is a tool for estimating the relative stability of natural slopes or landforms. LISA uses the infinite slope equation to compute the factor of safety against failure for a given set of site...
conditions (input values). The input values include ground slope, soil depth (depth to potential failure surface), soil cohesion, friction angle, unit weight, ratio of ground water to soil depth (Dw/D), root strength, and tree surcharge. For any particular input value, a probability distribution is used to represent natural variability and uncertainty in the value. LISA uses Monte Carlo simulation to sample the probability distribution of each input value and reports the probability of failure (PF#) as the percentage of factors of safety less than or equal to 1 (Hammond et al., 1992).

The two case histories below illustrate different ways of using LISA and other more subjective approaches to hazard mapping. Which is the most useful approach depends on the size of the area to be assessed, the desired degree of resolution of the potential failure site, the objective of the assessment, and whether repeatable numerical results are desired and how they will be used. Common objectives range from identifying areas needing more detailed analysis (level II for roads or level I with more refined map units for harvest) to estimating sediment production for cumulative effects analysis.

For sediment estimates to support land use planning, it may be necessary to use soil survey units. Forests having geologic resource and conditions maps (Reilly, 1989), such as the Gifford Pinchot National Forest, may use geologic materials map units (see Gifford Pinchot case history below). Useful map units for LISA analysis could be generated by subdividing these units by slope classes using a digital elevation model (DEM). For such large areas, geographic information system (GIS) support is necessary.

For assessments of future land use proposals, increased resolution is desirable for transportation planning or estimates of cumulative watershed effects. Map units can be refined to improve LISA estimates. Results can be used to compare probabilities among different areas and focus on data needed for level II.

Mapping directly from aerial photos using a subjective approach may be desirable for large-scale (5- to 50,000-acre), short timeframe projects, such as fire or forest health salvage (as in the Siskiyou case history below). National Environmental Policy Act (NEPA) needs will be met and specialists will need to spend more time in the field, but there will be less emphasis on collecting and storing site-condition data for future use.

Case History: Comparison of Subjective Hazard and Risk Mapping by Means of LISA, Siskiyou National Forest, OR (McHugh, 1991)

Methods: The probability of mass failure and risk of delivery to a stream channel was mapped on aerial photographs into relative classes of high, moderate, and low. The subjective mapping considered the following factors: ground slope, slope shape, proximity to stream channels, lithology, structure, existing landslides, landslide types, and location of failure-prone facilities (such as sidecast or stacked roads).

In the same 4,700-acre area, LISA was used to estimate the factor of safety for landforms mapped as bench, footslope, inner gorge, draw, or gulch (definitions modified from 1986 National Soils Handbook Glossary of Landform and Geologic Terms). Input values and their distributions were...
estimated from ground reconnaissance and correlations between areas of similar rock/soil type and geomorphology. The intensity of ground reconnaissance was low; approximately 3 miles per square mile were traversed. Because LISA input values and distributions are determined on a spatial basis, PF# can be interpreted as the percentage area of the landform expected to be unstable.

Results: Because the subjective mapping included the risk of sediment delivery to a stream as well as the stability of landforms, it was expected that the two methods might produce different results. However, the table below indicates that the percentage of a landform subjectively mapped as "high" compares well with the LISA probability of failure.

<table>
<thead>
<tr>
<th>Landform</th>
<th>Percent Mapped &quot;High&quot;</th>
<th>Probability of Failure in %</th>
<th>LISA Skyfall Factor of Safety</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bench</td>
<td>28</td>
<td>25</td>
<td>1.13</td>
</tr>
<tr>
<td>Footslope (Flat)</td>
<td>26</td>
<td>33</td>
<td>1.06</td>
</tr>
<tr>
<td>Footslope (Steep)</td>
<td>25</td>
<td>46</td>
<td>1.03</td>
</tr>
<tr>
<td>Draw</td>
<td>35</td>
<td>39</td>
<td>1.05</td>
</tr>
<tr>
<td>Inner Gorge</td>
<td>100</td>
<td>71</td>
<td>0.94</td>
</tr>
<tr>
<td>Gulch</td>
<td>90</td>
<td>51</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Discussion: The degree of resolution varies between the two approaches used above. The subjective map identifies sites within the LISA landforms which have lower factors of safety. For example, convergent (valley) and divergent (peak) slopes within the "steep" footslope landform are separated in the subjective mapping. For use with LISA, map units could be subdivided to a resolution approaching the subjective map. This approach would be useful where alternative road locations will be analyzed using level II or where alternative silvicultural methods or unit locations could be addressed using LISA.

When existing map units must be refined to improve resolution, LISA involves considerably more time than subjective mapping. As the user gains knowledge of soil mechanics, smaller areas can be targeted for analysis. LISA is expected to become more efficient as geotechnical soils data bases and GIS map layers are developed. Delineation of high resolution map units is the time-limiting factor in the analysis described above. If DEM’s could be used to map landforms into concave, planar, and convex elements, as well as to assign drainage area and slope distribution, some parameters could be automated. The benefits of this GIS approach will be limited by the
The dangers of automation are clear; no analysis can replace air photo and field verification.

Case History: Use of LISA on five third-order watersheds on the Gifford Pinchot National Forest, WA (Wooten, 1988)

Estimates from LISA of the proportion of land consisting of landslides under natural and harvested conditions were validated using ground reconnaissance. LISA can be used to characterize the stability of existing map units using input values from existing data bases. The Gifford Pinchot National Forest has developed a Geologic Resource Inventory with Geologic Resources and Conditions data base. However, these maps were not developed with sufficient resolution to assess effects of proposed specific harvest units. This project tested the results of LISA using various intensities of ground reconnaissance. The LISA probability of failure number (PF#) was compared with the number of natural and harvest-related failures and with the percentage of land in failure within the map unit.

Methods: Map units were delineated from aerial photographs based on ground slope, soil depth, material type, and soil depth-ground water ratio. Ground-slope distributions were estimated from a topographic base map (40-foot contour interval). Soil cohesion, friction angle, and soil depth were estimated initially from Unified Soil Classification values in the Geologic Resources and Conditions data base. Root cohesion was related to the tree root-substrate classification of Tsukamoto and Kusakabe (1984). Ground reconnaissance was used both to validate the percentage of land in failure (air photo estimates were 1 to 3 percent low) and to modify values of soil cohesion, friction angle, and depth. Three ground water methods were used:

Ground Water Method 1: Potential ground water concentration and dispersion were assumed to be related to landforms. Values were chosen from the literature to represent the areal distribution of soil depth-ground water ratio (Dw/D) at the annual maximum for the period of minimum root cohesion resulting from at least one "major" rain-on-snow event.

Ground Water Method 2: Dw/D was back-calculated using the percent of land in failure on selected map units to match the PF#. This Dw/D was tested on similar landforms, adjusting for the variation of Dw/D with soil depth.

Ground Water Method 3: A peak Dw/D was calculated for an existing failure. The percentage of the surrounding map unit expected to have this Dw/D was estimated. These values were tested on an existing harvest unit.

Results: The rankings of the map units based on PF# corresponded with numbers of natural and harvest-related failures. The correlation between PF# and percentage of land in failure depended on the intensity of ground reconnaissance and the ground water method used. Sources of error were in slope estimates from the map base and the minimum root cohesion value.
after harvest (not yet achieved in recently harvested units). Method 1 gave PF#'s that were too high, but they were improved by using method 2. Method 3 over-predicted the percentage of land in failure on the harvest unit, but this was explained as an error in the estimate of the area subject to the peak Dw/D.

Discussion: Ground reconnaissance is necessary to achieve the desired resolution for evaluating stability of proposed harvest units. Field verification of input values and location of land in failure for back-calculation of ground water values improves the accuracy of the LISA PF#.

Where rotational failures are present, LISA may be used if input variables are selected for the center of gravity (Ristau, 1988). Hammond et al. (1992) found that LISA may overestimate factors of safety where soils are less than 2 feet deep. The soil-ground water interactions controlling slope failure, surface erosion, and piping processes may not be well understood under these conditions.
Consideration should be given to natural rock slopes in valleys that have a history of stress relief and those associated with layered rock formations of differential strength. Valley sideslope collapse and resulting talus and deep colluvial toe slopes can be the result of valley stress relief and elastic rebound (Matheson and Thomson, 1973). Valley stress relief and the resulting rebound is a function of the elastic modulus of the rock, deviator stress, and cyclic intervals. Stress application to valley sidewalls and floors can result from alpine glaciation or outwash sediment accumulation. In alpine glacial environments, these events are usually recompression cycles, as a function of glacial advance and retreat in response to global warming and cooling trends. This cyclic deviator stress tends to have a more adverse effect on rock slope stability than single valley-stage alluvial development by erosion and deposition.

In alpine glacial sequences, lateral and vertical compression is applied to the valley during glacial advance. Stress removal takes place during glacial retreat, which causes buckling in the side slopes, defined as zones of extension or tension, and arching in the valley floors, defined as zones of compression (Ferguson and Hamel, 1981). Young and Shakoor (1987) completed a study of valley stress-relief-induced slope failures along the Ohio River in West Virginia and Ohio. They found that the most prominent joint sets in this area were those paralleling the river and were a result of release of residual stress, normal to the valley, caused by river downcutting.

In their study of valleys in western Canada, Matheson and Thomson (1973) recognized that upwarping of the valley floor influences the dip of bedding in sedimentary rocks and may produce interbed slip and gouge zones that influence slope stability when they are located at the toe of slopes. They also concluded that valley rebound (strain) may be up to 10 percent of the valley's depth, but more typically is found in the range of 3 to 5 percent.

Minor structural features may be an indication that valley sidewalls are in tension. Features such as minor anticlinal structures in valley floors, inflated or “diced” rock masses caused by cooling joints in igneous extrusive rocks being intersected by stress relief joints, near-vertical stress relief joints parallel to the valley, tension cracks, and overhangs are all indicators of stress relief or differential weathering. Natural processes which can accelerate the rates of failures in these terrains include freeze-thaw cycles, wind-levering of trees, root-wedging, elevated pore water pressures in discontinuities, and rapid drawdown of reservoirs.
Differential weathering occurs in rock masses that are layered with sequences of alternating rock of different strength, such as sandstone and shale sequences, basalt flows over glacial till, or andesite flows with pyroclastic interbeds. In these situations, the lower-strength material weathers at a faster rate than the overlying material, resulting in an overhang. This force imbalance eventually results in failure of the overhang, which may precipitate mass failure of the slope (Arambarri and Long, 1993).

Wide-area analysis of structural discontinuities may begin in the office, using data obtained from geology maps and aerial photo interpretation. The trend and plunge of anticlines and dip/dip-direction of the limbs can be plotted on an equal-area stereo net as poles or dip vectors. The trend of the valley and magnitude and direction of the valley sideslopes can then be plotted as great circles. Faults, bedding, and foliation may also be plotted, forming a complete data set of any particular section of the valley, and used in a Markland stereo net analysis for kinematic failure potential (see section 3C.2 for a complete discussion of this method). Areas that indicate high potential for failure should then be examined on the ground.
2E. Cumulative Effects

2E.1 Cumulative Watershed Effects and Land Use Planning

2E.1.1 Introduction

Cindy Ricks, Resource Geologist, Siskiyou National Forest
Thomas Koler, Research Engineering Geologist, Intermountain Research Station

The Council of Environmental Quality Regulations for Implementing Procedural Provisions of NEPA defines "cumulative impact" as "the impact on the environment which results from the incremental impact of the actions when added to other past, present, and reasonably foreseeable future actions regardless of what agency (Federal or non-Federal) or person undertakes such other actions. Cumulative impacts can result from individually minor but collectively significant actions taking place over a period of time" (U.S. Congress, 1969).

The legal definition of cumulative effects is broad. NEPA regulations require that both positive and negative impacts be analyzed. Because the term "impact" has developed an association with primarily negative effects, the more neutral term "effect" is often used.

In watersheds, effects may occur at locations distant from land management activities (offsite effects) and at times apparently unrelated to such activities (indirect effects). Cumulative watershed effects concern water-related resources that may be affected by changes in water quality, amount and timing of streamflow, and stream channels. Stream channel changes may be caused by increased sediment loads from landslides or surface erosion, changes in supply of large woody debris or boulders, or modifications of basin hydrology, such as increased peak flows. Potential changes in channel conditions include: increased fines in gravel, increased cobble embeddedness, filling of pools, channel widening, and loss of cover and refuge from winter high-flow or summer low-flow conditions, all of which affect the quality of fish habitat.

Land management practices also have the potential to affect stream temperature, nutrient levels, and turbidity, as well as flooding and low flows (Ice et al., 1990). Reduced low flows, overall reduction of water yield, and changes in timing of flows are important concerns for irrigation (McDonald, 1990).

In addition to multiple forest management activities through time, non-forest activities—such as agricultural, residential, and urban development—are considered in a thorough analysis of cumulative watershed effects. The responsibility for assessing cumulative watershed effects generally resides in the soils/watershed/range staff area. Geologists and geotechnical engineers can support this assessment with the quantitative and qualitative tools and methodologies described in this guide.
Cumulative watershed effects may be viewed as a chain of causes and effects, where each link or process is subject to modelling with varying degrees of certainty. Where causes and effects are not well understood or data are insufficient, empirical correlations are often used rather than process-oriented models (Prellwitz, 1993). For example, the volume of landslide sediment produced during the last decade may correlate with turbidity at a particular flow, but processes of sediment delivery and transport of fine sediments are ignored in this correlation.

On national forest lands, sediment transport is dominated by colluvial and alluvial processes, with volcanic, eolian, lacustrine, and coastal processes found locally. Mass wasting of soil and rock materials and surface erosion are the mechanisms of sediment transport on hillslopes, while debris flows and dam-break floods (debris torrents) are those found within stream channels (Benda and Miller, 1991).

The several processes and materials involved in slope movement produce a variety of movement types (Selby, 1990). Varnes' (1978) classification of landslides describes both process (mechanism of movement and mode of deformation) and rate (velocity).

Hillslope-channel linkages vary across the landscape with geomorphic setting. Patterns of landslides and associated hillslope processes have direct and indirect effects on stream channels. Effects on the geometry and disturbance regime of channels and streamside areas are expressed in valley-floor landforms and stream ecosystems (Swanson et al., 1987). Ecosystem processes, such as rates and patterns of succession and nutrient cycling, are affected.

Frequency, persistence, and mechanisms of stream sediment transport are determined by interaction of flow regimes with the abundance, timing, and size distribution of sediment delivery. Landslides also affect delivery, redistribution, and removal of large woody material in streams providing habitat structure and diversity (Swanson et al., 1982).

Natural and management-related disturbances may differ in their types, frequencies, intensities, and extent of disturbance. Timing of triggering events relative to disturbance (such as wildfire, road construction, or timber harvest) may result in brief periods of dramatically increased sediment production (Swanson et al., 1982). More frequent disturbance of channel and streamside areas in response to triggering events is the expected result of accelerated disturbance under management.

Hillslope-channel linkages are strongly influenced by relative rates of hillslope sediment delivery as compared with channel export. These linkages may occur over a landscape/stream network or a stream reach. Caine and Swanson (1989) describe two mountainous terrains with contrasting equilibrium and decay states of landscape evolution. Landslide deposits resulting from slope failures are typically delivered to streams in increments, rather than as a single event. Catastrophic failures tend to be associated with natural events, such as earthquakes and volcanism, and may result in river blockage and valley filling (e.g., the Toutle River in western Washington during the May 18, 1980, eruption of Mount St. Helens).

Benda and Dunne (1987) and Benda and Cundy (1990) characterized debris-flow initiation, runout distance, and stopping location across a landscape/stream channel network. Grant et al. (1984) described a method for assessing channel response to
2E.1.4 Sediment Budgets/Sediment Yields

The sediment budget synthesizes what is known about cause-effect relations and hillslope-channel linkages. Dietrich et al. (1982) defined a sediment budget as follows:

A sediment budget for a drainage basin is a quantitative statement of the rates of production, transport, and discharge of detritus. In most studies, measurement of sediment production is assumed equivalent to quantification of sources of sediment discharged into streams. Sediment is also produced by the chemical degradation and physical mixing of weathered bedrock to form a soil, however, and material is transferred between size fractions as a result of attrition during transport. In a sediment budget, the soil mantle should be treated as a separate element of sediment storage having a definable rate of inflow and outflow detritus. Quantification of debris transport requires not only defining rates of sediment transfer between major storage elements—such as between soil and stream channels—but also computing rates of movement through these sediment reservoirs. The latter is particularly difficult to do with the present understanding of processes, but it is from such a quantification that predictions can be made about the change in size of storage elements and the rapidity and magnitude of the response of sediment discharge from a drainage basin after some disturbance. Discharge of detritus is simply the rate of transport of sediment past a monitoring station.

To construct a sediment budget for a drainage basin, the temporal and spatial variations of transport and storage processes must be integrated; to do so, these requirements should be fulfilled: (1) recognition and quantification of transport processes, (2) recognition and quantification of storage elements, and (3) identification of linkages among transport processes and storage elements. To accomplish this task, it is necessary to know the detailed dynamics of transport processes and storage sites, including such problems as defining the recurrence interval of each transport process at a place.

Sediment yield, frequently confused with sediment budget, is defined by Swanson et al. (1982) as part of an analysis of an individual erosion process, including the
2E.2. The Role of Stability Analysis in Cumulative Effects Analysis

Thomas Koler, Research Engineering Geologist, Intermountain Research Station

2E.2.1 What Are Cumulative Effects?

Cumulative effects analysis (CEA) is a commonly misunderstood term. Some prefer to substitute the word “impact” for “effects” because that seems to make more sense. Others prefer the title “watershed cumulative effects (impact) analysis,” and some simplify this with the names “watershed analysis” or “basin analysis.” Our colleagues (Ice et al., 1990, p. 2) have defined cumulative effects as:

... various on-site and off-site impacts of multiple forest management activities on resource values through time. Cumulative watershed effects (CWE) can include any changes in water quality, amount and timing of streamflow or impact to the stream channel which affect beneficial uses. Currently, most cumulative watershed effects concerns are about changes in channel conditions resulting from either increased sediment loads from mass-wasting or surface erosion, change in in-channel obstructions (large woody debris) or modification to basin hydrology such as increased peak flows. Potential channel conditions changes related to these concerns include: increased fines in gravel, increased cobble embeddedness, filling of pools, channel widening, and loss of cover and refuge from winter high-flow or summer low-flow conditions, all of which can influence the beneficial use of streams for fish habitat. Other cumulative watershed effects about which forest managers can and have been challenged include changes in stream temperature, nutrient levels, and turbidity, as well as concerns about flooding and low-flows.

From the perspective of the Forest Service, this definition is not completely accurate. For example, there are many non-forest management activities that affect resources, such as those in areas containing a patchwork of different landowners within a river basin. Agriculture, residential and urban development, and non-forest recreation areas are some examples. Other environmental changes include seasonal changes and general reductions of water yield. These changes are important to irrigators (McDonald, 1990). Also, an increase in sediment flux into a fluvial system is not always an adverse effect; in some cases, an increase in the stream bedload can benefit fish and other biota.

So why is this a concern for us in engineering, and specifically in geotechnical engineering? Our colleagues in the soils/watershed/range staff certainly have the responsibility to work in this arena; however, we have the professional abilities to support them with the tools and methods described in this guide. Frequently our
colleagues use qualitative methods in their slope stability assessments. This is a scientifically valid process, but we have quantitative tools to reinforce the qualitative results which we can (and should) share with these colleagues.

2E.2.2 Elements in the CEA Process

I. Science

Engineering geologists by definition have three scientific skills: we can identify the origin of materials, describe the history of geologic events, and categorize them by geologic processes (Williamson, 1979). This science is an important element in the CEA process pertaining to slope stability and sediment transport.

Origin of Materials

In most Forest Service CEA models there are two components that can be identified by the origin of materials. These are materials transported and deposited by means of natural (geologic, climatic, and fire) events and cultural (logging, roading, range, and fire) processes. In a sense, geologists have been involved in identifying natural events by means of CEA since Hutton presented the theory of uniformitarianism (Hutton, 1795). We understand the relationships among mechanical and chemical weathering, erosion, transportation, and deposition of sedimentary materials. Engineering geologists have evaluated depositional histories of transported sediments resulting from cultural events since the 1920’s, when Terzaghi took the lead in developing modern engineering geology.

As we modify the surface of the Earth by road construction, road obliteration, and other management practices, we have the methods to identify and evaluate these disturbed materials. By evaluating the cause-and-effect relationship of what moves where, we can predict which materials will move downslope. By knowing the process and rate, we know how these materials move and when movement will occur. This becomes very important when the materials move downslope in large volumes and severely affect resources. By identifying and describing the origin of materials and how they are moved and deposited within a basin of interest, the engineering geologist can develop a sediment budget for input into the CEA model.

History

A typical point of reference in sediment transport CEA is what is colloquially referred to as “background levels.” This term commonly refers to natural hydrologic and geomorphic conditions. In other words, what is the condition of the watershed in an undisturbed state at a moment in time? This background level is then used for comparing predicted conditions in the CEA model. The major fallacy in this concept is the failure to recognize that landforms evolve over geologic time. What is perceived to be a large flux of transported sediment from a particular management activity might be surpassed by a larger flux from natural hydrologic and geomorphic processes. It all depends upon the specific time on the geologic clock.

Rather than using supposed background levels, we should provide a history of the watershed of interest. If a few decades worth of aerial photographs are available, we can take advantage of these tools to develop a recent history. For a geologic history beyond the most recent time (say the last 50 years), we can work with soil scientists to establish the timing and location of large fires and for development of the soil
profile. By interpretation of geologic and seismicity maps (available through the State geological surveys and the U.S. Geological Survey), we can estimate the location of possible recent fault activity and associated geomorphic features (rock avalanches, alluvial fans, etc.). Hydrologists and climatologists can estimate the climatic history for the last few centuries. By describing the geologic history we can assist CEA modelers in developing more realistic natural sediment yields.

**Process**

On national forest lands, the dominant processes in sediment transport CEA are colluvial and alluvial. There are also the rare exceptions of volcanic, aeolian, and lacustrine-coastal processes. Sheetwash and the mass-wasting of soil and rock materials are the mechanisms of sediment transport on hillslopes, while debris flows and dam-break floods (debris torrents) are those found within stream and river channels (Benda and Miller, 1991). It is very important to understand the hydrology; for example, what are the relationships among the landform evolution, slope stability, and surface/subsurface flow (Horton overland flow, subsurface flow, and return flow)? By putting these process pieces together, we can be descriptive in explaining what is occurring within a basin of interest.

Probably the most common classification of landslide types and processes used by Forest Service engineers and engineering geologists is the one developed by Varnes (1978). However, there are other classifications that are useful (e.g., Hutchinson, 1968). Varnes' classification is divided into two types of materials—bedrock and engineering soils. The soils are further divided by the dominant grain size (coarse-grained or fine-grained) and by the moisture content. In this classification system the materials for flows are described by rate of movement. The advantage of this method is that it incorporates in descriptive fashion the process rate of the landslide.

II. How Analysis for Cumulative Effects Fits into Forest Service Planning

With the plethora of sediment transport models available to Forest Service specialists, it is no wonder that confusion and lack of uniformity pertaining to CEA are common in our agency. Despite this situation, we do have a clear framework to work within; for example, NEPA and the National Forest Management Act (U.S. Congress, 1976) provide the policy to follow in this process. A process that has been successful for the Olympic National Forest uses this framework for a sediment transport CEA (Webster and Koler, 1991). The limiting factor in this process is not the model used but the legislated policy; therefore, no specific sediment transport CEA model is required to make this process successful. The following is extracted from Webster and Koler (1991).

**Forest Plan**

Figure 2E.1 shows how planning begins and ends with the forest plan. The equivalent levels in the sediment transport CEA process start with a sediment yield model—such as R1–R4 (Cline et al., 1981) or WATSED (1991)—and end with monitored data input to the forest GIS. These sediment-yield models are not quantitative, and in most cases the values are very subjective. However, the information at this level is appropriate for a first approximation of what is occurring within the river basin (typically at a mapping scale of 1:63,360).
Integrated Resource Planning

Figure 2E.1.—Forest plan process.

10-Year Scheduling

In creating a 10-year activity schedule, district rangers decide in which drainage basins management activities will be conducted and how much activity will occur in each basin. These managers need to know the condition of each basin in order to minimize adverse cumulative effects and to document the need for rehabilitation of highly damaged areas. Sediment-yield models can also be used for this planning level; however, a more accurate method is to calculate the sediment budget for the basin of interest, as summarized in section 2F. These sediment budgets will vary depending on the proposed management activities within the basin. Sediment budgets calculated for several years prior to the current scheduling are a great help in understanding the most recent geologic processes occurring in response to "natural" and "cultural" events. The typical map scale for this 10-year scheduling work is 1:24,000.

Project-Level Planning

Scoping

Analysis areas are identified during the scoping stage—boundaries are specified and the presence and condition of the resources in the area are inventoried. Issues, concerns, and opportunities can be defined from this information. If the analysis area has sediment yields or budgets that reach or surpass an arbitrary predetermined threshold, a hazard-risk assessment will quantify the erosional processes. A hazard is
an existing or potential erosional feature or condition that has the potential to contribute sediment to the fluvial system. A risk is a resource (e.g., fish and wildlife habitat) that has the potential to be degraded in response to hazards.

This assessment is commonly completed by evaluating surficial erosional processes (e.g., rainsplash and sheetwash), soil (regolith) slope movement (e.g., translational and rotational landslides), and rock slope movement (e.g., rock falls, topples, and slides). Several tools available to quantify the cause-effect and process-rate of these hazards are: the Modified Universal Soil Loss Equation (USLE)* for surficial erosion, LISA (Level I Stability Analysis, 1991) for soil translational failures, SARA (Stability Analysis for Road Access, in process) for soil rotational failures, the Benda-Cundy Model (1990) for debris flows, and the rock slope engineering method developed by Hoek and Bray (1981). The typical scale for this level of work is 1:24,000 or 1:15,840. Section 2F contains a more complete description of how this assessment is put together. The hazard-risk assessment can be completed during scoping, but is probably most effective if used at the alternative-formulation stage of the planning process.

Formulation of Alternatives

During alternative formulation, the hazard-risk assessment can be done for all or part of the analysis area. This assessment can be used by the district planner to concentrate or avoid management activities on specific sites and consequently to be proactive in predicting the degree of impact. This should be done on lands where the sediment-yield model or sediment budget calculation indicates a cause for concern. This step allows the district ranger to control or mitigate the hazards that put resources at risk as a result of the proposed activities.

Assessing Effects

Predicted volumes of transported sediment for each alternative can be estimated by sediment-yield models or calculated from sediment budgets. The materials that are delivered to the fluvial system can be evaluated as they are routed if the channel geometry, gradient, average particle size ($D_{50}$), roughness, and 2-year and 100-year flows are known. Bed-material flux can then be calculated to determine scouring or aggradation effects on the stream channel for individual stream segments of interest. By predicting the sediment volumes delivered to the fluvial system for each alternative, and by routing the bed material through the system under different flows, the engineering geologist is able to provide valuable input to the interdisciplinary team preparing the environmental analysis or environmental impact statement.

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* Editor's Note: The Universal Soil Loss Equation (USLE) is being replaced by Water Erosion Prediction Project (WEPP) computer programs for estimating soil erosion (Elliott, 1994). The WEPP programs, under development since 1986, are based on field testing and modeling of forested lands, harvested lands, and roads in addition to agricultural and rangeland conditions. The first forest applications of the WEPP model are being developed in 1994, as this guide is going to press. The WEPP programs should be considered as potential replacements for the USLE discussed in this section.
Decision

Once an alternative has been selected, a forecasting routine can be run to estimate the time at which the sediment levels will drop to the point such that further activities in the analysis area can be planned. Currently, all of this analysis can be completed using a personal computer; in the very near future, most calculations can be performed within GISes. Therein lies a problem: a scientist can “complete” the analysis in the office without verifying the data by field work. Therefore, it is critical that a peer or a senior scientist reviews this work before providing the information to the district ranger or other line officer.

Monitoring

Management activities seldom occur as planned; hence, the results of the forecasting routine will have to be updated as activities occur. One effective monitoring method is simply to update the inventory of the geomorphic and hydrologic processes within the basin of interest. This includes a landslide inventory, climate record, and changes in the fluvial geomorphology (geometry, gradient, bedforms, and bed material size). Monitoring the fluvial processes may at first seem an intensive process. To calculate areas of scouring and aggradation effectively, the bed substrate must be evaluated to calibrate the flux of material into and out of the stream segment of interest. This can be done in a couple of field days for a river basin approximately 8 square miles in area. Monitoring field work includes the observation of changes in the fluvial system, sketches, and general comments. Also, a review of the aerial photographs of the area over a few decades helps to better understand previous fluvial processes that responded to hillslope processes similar to the ones you are monitoring. These data are put into a GIS or other data base system for quick retrieval. The few days invested in keeping these monitoring data updated will minimize future management problems.

Forest Plan

The planning process ends at the forest-plan level. Monitoring information and documentation supporting previous decisions are important, especially if amendments will become necessary. These annotations are important because the geology of an area is not static over time.

2E.2.3 Need for High-Quality Input to the Stability Analysis Process

Understanding the geomorphic and hydrologic processes is a first step in providing input to the sediment transport CEA. The next step is to quantify this understanding and place it in a framework, the results of which will be a product that land managers can use to make decisions based on good science. If the earth scientist can explain what is creating an unstable slope (cause), where it will fail and move to (effect), how it is moving (process), and when it will get to an area of interest (rate), he or she will have a good start in telling the sediment transport story. The other part of this story is the dimensions. What is the volume of material in storage that has the potential to be transported downslope into a fluvial system? What is the overall sediment budget for the watershed of interest, and where are the critical input areas where the sediment yield will be detrimental to resources such as fish habitat and municipal water supply (hazard-risk)? This second part of the story interests land managers the most; however, to get this part, the earth scientist needs to complete the first part: sediment yield.
Cause and Effect of Slope Failures

Cause-and-effect descriptions of unstable (or potentially unstable) slopes provide answers to the "what" and "where" questions. Causes for unstable slopes are based in geology, soil and rock mechanics, and hydrogeology. Many engineers and geologists have described these causes (e.g., Ladd, 1935; Sharpe, 1938; Bendel, 1948; Terzaghi, 1950; Rapp, 1960; Legget, 1962; Zaruba and Menc, 1969; Skempton and Hutchinson, 1969; Krinitzsky and Kolb, 1969; Sowers, 1979; Dunn et al., 1980; Costa and Baker, 1981; Selby 1990).

Varnes (1978) has probably provided the most lucid summary of unstable slope causes, which he lists under two categories: factors contributing to increased shear stress and factors contributing to low or reduced shear strength. The location at which the slope might fail and how it will travel downslope best define what the effects will be for an unstable or potentially unstable slope.

In many cases, the geomorphic feature that is moving does so incrementally, rarely reaching the fluvial system in one movement. However, there are catastrophic examples of large slide masses moving thousands of feet downslope and into a stream channel. These can occur in response to management activities—such as road construction and timber harvest—but are more frequently the result of sudden natural events, such as earthquakes, volcanism, heavy rainfall, or snowmelt. A typical effect of these catastrophic events is river blockage and valley filling (e.g., the Toutle River in western Washington during the May 18, 1980, eruption of Mount St. Helens). In the more typical case, where the slide mass is much smaller and moves incrementally, the usual result is a small increase in the sediment yield at a particular point in a river system. This increase may degrade fish habitat or some other resource. In the cumulative-effects perspective, however, if you have several of these typical slide masses moving incrementally downslope over a period of time, the total sediment budget can be quite large, approaching a rate (tons/mi²/yr) similar to one large and sudden event. Earth scientists, then, need to explain where a landslide feature will travel and what the consequences will be as it moves downslope.

Process-Rate

An understanding of the process and rate of slope movement answers the "how" and "when" questions of slope stability. There are several processes and materials involved in slope movement with a consequent variety of movement types (Selby, 1990). Therefore, there are several criteria to be followed in classifying these features: velocity and mechanism of movement, material, mode of deformation, geometry of the moving mass, and moisture content. Varnes’ (1978) classification of landslides covers both process and rate and is the system used by most Forest Service engineering geologists and geotechnical engineers.

As described earlier in this section, Varnes’ classification is divided into two types of materials: bedrock and engineering soils. The soils are further divided by the dominant grain size and relative moisture content. Types of movement are falls, topples, rotational and translational landslides, lateral spreads, flows, and complex movements. By using this classification, both the process (mechanism of movement and mode of deformation) and rate (velocity) can be effectively described.
Geomorphic Watershed Analysis

In a geomorphic watershed analysis, cause-effect and process-rate need to be integrated within a sediment budget approach that also includes a product for management to use in making decisions. This product should be a hazard-risk assessment for the analysis area of interest. In a hazard-risk assessment, the geomorphic processes are evaluated for a particular analysis area (such as a river basin) at a given time and it is determined how these processes may affect the resources in or downstream from the area. As defined above, a hazard is a geomorphic feature or process that presently is, or has the potential to become, unstable. At risk is the resource (e.g., timber, water quality, fish and wildlife habitat) that may become degraded in response to a hazard condition. Therefore, the hazard-risk analysis is a two-step method: (1) Identify the hazard and the condition over time and (2) Identify the resources susceptible to degradation over time as the result of the hazards.

Identifying the hazards and risks is only part of this geomorphological watershed analysis. To get a complete picture of what is occurring in the analysis area, one must link through hillslope processes, over time, the way material is stored, transported, and delivered to a fluvial system. Once the material enters the fluvial system, the suspended and bedload sediment being routed through the system must be evaluated. The first step then is to develop a sediment budget for the analysis area incorporating the hazard-risk assessment. Using this sediment budget, one can then calculate where material will travel during a particular timeframe and how it gets into the fluvial system and is routed downstream. In essence, one identifies the geomorphic processes for a timeframe and points to the “pathway” by which materials move downslope and downstream.

Providing Forest Service management with a hazard-risk assessment report is not the only step in assisting management with their decision making within the CEA process. Managers need predictions, based on good scientific methods, of what will happen cumulatively in response to a proposed alternative. In association with these predictions, they need some options for mitigating areas of increased sediment-transported materials. For example, different types of engineered structures may be viable for slowing and stopping sediment (for the lifetime of the structure, typically 25 to 50 years); living “structures” within the current state of biotechnology can provide similar structures (usually with a lifetime of several decades). An analysis area which is above background levels can still be managed; in fact, it may make good ecological sense to manage the basin to stabilize and diminish the sediment production and transport there. However, to provide this type of recommendation, we must know our facts and assumptions and follow a scientific process that quantifies the geomorphic processes in the analysis area.

The three-level system is a viable means that the geotechnical specialist can use in evaluating cumulative effects within the planning process. This system provides a quantitative analysis that enables a good understanding of the cause-effect and process-rate of active and potential slope movements. Using this system, we can delineate hazards and risks and provide empirically and stochastically based data for CEA pertaining to slope stability.
2F. Watershed Analysis Case Histories

Cindy Ricks, Resource Geologist, Siskiyou National Forest
Juan de la Fuente, Forest Geologist, Klamath National Forest

Geomorphic watershed analysis uses a sediment budget approach to provide a hazard-risk assessment. The hazard-risk assessment identifies what geomorphic processes operate in the watershed and how these processes may affect the resources in or downstream from the analysis area. The hazard-risk analysis has two parts: (1) Identify the hazard and the condition over time and (2) Identify the resources susceptible to degradation as the result of the hazards.

To quantify these hazards, several tools are available: the Modified Unified Soil Loss Equation (USLE)* for surficial erosion, LISA (Level I Stability Analysis, 1991) for soil translational failures, SARA (Stability Analysis for Road Access, in process) for soil rotational failures, Benda-Cundy (1990) Model for debris flows, and the rock slope engineering methodology developed by Hoek and Bray (1981).

Quantifying hazards and risks is only part of the geomorphic watershed analysis. Hillslope processes must be linked over time to the way material is stored, transported, and delivered to a stream channel. The next step is to develop a sediment budget for the analysis area incorporating the hazard-risk assessment. Using this sediment budget, you can calculate where material will travel for a particular timeframe and how it gets into the stream channel and is routed downstream. Suspended and bedload sediment transport rates are evaluated.

A typical point of reference in the analysis of cumulative watershed effects is the natural or undisturbed state (sometimes known as baseline or background level). Because landforms evolve over geologic time and disturbance is a natural process (fire, flood, climatic change), a baseline hydrologic or geomorphic steady-state condition may not exist. The condition of a watershed prior to disturbance from management activities at a particular time may be assessed.

For a historical perspective prior to aerial photography (generally covering the last 50 years), ecologists and soil scientists may have data on timing and location of large fires and development of the soil profile. From interpretation of geology and seismic

* Editor's Note: The Universal Soil Loss Equation (USLE) is being replaced by Water Erosion Prediction Project (WEPP) computer programs for estimating soil erosion (Elliot, 1994). The WEPP programs, under development since 1986, are based on field testing and modeling of forested lands, harvested lands, and roads in addition to agricultural and rangeland conditions. The first forest applications of the WEPP model are being developed in 1994, as this guide is going to press. The WEPP programs should be considered as potential replacements for the USLE discussed in this section.
activity maps (available through State geological surveys and the U.S. Geological Survey), recent fault activity and associated geomorphic features (e.g., rock avalanches, alluvial fans, etc.) may be located. Using these data with historical landslide inventories and the climatic history, sediment yields may be estimated.

**Case History: Salmon River Basin Sediment Analysis, Klamath National Forest, CA. (Data from de la Fuente and Hessig, 1993.)**

**Geomorphic Terranes**

The Salmon River is tributary to the Klamath River and occupies 744 square miles in the central part of the Klamath Mountains in northwest California. The geomorphic terranes used in this study were defined to discriminate between areas of characteristic slope processes and landslide production rates. Some terranes were found to produce most of the landslide volume. In addition, the portions of these terranes in which roads were constructed, or which were subjected to vegetation removal, exhibited accelerated landslide rates.

**Climatic Events**

Historic accounts reveal that the latter half of the 19th century was generally wet in the Pacific Northwest and that large floods occurred in the winters of 1861–62 and 1889–90. From 1915 to 1950 precipitation was low in the Salmon River watershed, and peak discharges in the river at Somes Bar on the Salmon River were also low. This dry period was followed by a wet period from 1950 to 1975 during which floods were frequent. The largest of these occurred in 1964; this flood was probably larger than the 1890 event but smaller than the 1861 event (Coghlan, 1984). Since 1975, the climate has been primarily dry, with the exception of very wet winters in 1981–82 and 1982–83 and a wet spring in 1986.

**Sediment Production from Landslides and Soil Erosion**

1. **Salmon River:** During the period from 1944 to 1988, approximately 10.4 million cubic yards of landslide-derived sediment entered the stream system. Of this, 58 percent was associated with the 1964 flood and 17 percent with the storms of 1965–75. Undisturbed land accounted for most of this volume (67 percent), while roads accounted for 30 percent, timber harvest 2 percent, and fire 1 percent. During this same period, surface erosion produced less than 1 million cubic yards of sediment. Channel erosion was not measured quantitatively, but, in most cases, was observed to be directly related to storage of landslide deposits. Channel storage probably contributed an additional volume of several million cubic yards of sediment to the total amount moved during the period of this investigation.
2. Tributaries: Landslide production for the main tributaries to the Salmon River from 1944 to 1988 was as follows (in cubic yards):

<table>
<thead>
<tr>
<th>Tributary</th>
<th>Volume (cubic yards)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Stem Salmon</td>
<td>4,392,458</td>
</tr>
<tr>
<td>North Fork Salmon</td>
<td>1,411,289</td>
</tr>
<tr>
<td>South Fork Salmon</td>
<td>2,610,519</td>
</tr>
<tr>
<td>Wooley Creek</td>
<td>1,997,811</td>
</tr>
</tbody>
</table>

3. Large Landslides: Two large landslides created temporary dams on the main stem of the Salmon River and accounted for about 20 percent of the total landslide volume measured by this study. These landslides greatly affect projections for future landslide production.

History of Watershed Disturbances

1. Mining: Hydraulic mining of the main river valleys occurred from about 1880 to 1950 and involved at least 1,200 acres of land. Preliminary estimates of volume indicate that about 15 million cubic yards of sediment were discharged into the river during this time, and major channel modification occurred in many areas, particularly along the upper South Fork of the Salmon River.

2. Roads: There are about 3,620 acres of road in the watershed, two-thirds of which were built after 1955. There are about 4 acres of road per mile of length.

3. Timber Harvest: The earliest timber harvest occurred in conjunction with mining and homestead activity. Regeneration harvesting of timber on public land did not begin until the 1950’s. By 1975, there were about 8,000 acres of harvested public land (plantations) in the watershed and, by 1990, about 30,000 acres. The 1990 figures include about 18,000 acres of harvested land burned by the fires of 1977 and 1987.

4. Fire: Large fires involving about 70,000 acres occurred early in this century (1917 and 1918). No data are available on the intensity of these fires. From 1919 to 1976, the burned area was small, with only two fires larger than 5,000 acres. In 1977, 57,000 acres burned in the Hog Fire, and 102,000 acres burned in the wildfire of 1987. There were 20,000 acres of overlap between these two fires. Together, the area of land burned at high and moderate intensity was about 50,000 acres.

5. Total Disturbance Area: In 1975, timber harvest, road, and hydraulic mine disturbance covered only 2.5 percent of the watershed area. Following the fires of 1977 and 1987, the area disturbed by fire, harvest, road, and hydraulic mining covered 18.7 percent of the watershed. Substantial revegetation (about 8,000 acres) has occurred on the land harvested prior to 1975 because these plantations are now from 18 to 30 years old. Similarly, some of the hydraulic mines have revegetated.
1. **Landslide Rates:** Roaded lands produced landslides at a rate 24 to 100 times greater than undisturbed lands. This was mainly a function of excavation or fill placement during road construction that changed the existing slope geometry and/or ground water regime enough to unbalance driving and resisting forces on slopes that were at equilibrium. Harvested lands increased landsliding by 2 to 11 times. For example, during the 1965-75 period, each acre of harvested land produced 7 cubic yards of landslide material, while each undisturbed acre produced 3 cubic yards. Data are insufficient to distinguish the change in landslide rate associated with fire from that of harvest. Table 2F.1 displays landslide rates for undisturbed, roaded, and harvested land. Separate rates are given for the 1964 flood and for the combined floods of 1965-75.

<table>
<thead>
<tr>
<th>Type of Disturbance</th>
<th>Landslide Rate (yd³/acre)</th>
<th>Increase Over Undisturbed Rate (Times)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1964 Flood Event</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roads</td>
<td>909.4</td>
<td>108.2</td>
</tr>
<tr>
<td>Harvest</td>
<td>96.8</td>
<td>11.5</td>
</tr>
<tr>
<td>Undisturbed</td>
<td>8.4</td>
<td>1.0</td>
</tr>
<tr>
<td><strong>1965-75 Flood Events</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roads</td>
<td>158.0</td>
<td>59.4</td>
</tr>
<tr>
<td>Harvest</td>
<td>6.8</td>
<td>2.6</td>
</tr>
<tr>
<td>Undisturbed</td>
<td>2.7</td>
<td>1.0</td>
</tr>
</tbody>
</table>

One large landslide that occurred in 1964 delivered about 1 million cubic yards of sediment to the river; this slide has had a large effect on the road-associated landslide rate. If this slide is removed from consideration, the road rate decreases from 909 to 199 cubic yards per acre, and the increase over natural decreases from 108 to 24 times.

2. **Surface Erosion:** The universal soil loss equation (USLE), which was used to predict sedimentation associated with surface erosion, assumed that the existence of roads accelerated erosion by a factor of 6,000 to 9,000 and newly harvested units by a factor of 400 over undisturbed conditions. Surface erosion was found to be a severe problem on burned granitic lands. Following the 1987 fires in northern California, large volumes of sand were delivered to the Salmon River from burned granitic watersheds. Sedimentation rates as high as 4 cubic yards per acre per year were observed in spite of the fact that winters were relatively mild. Average annual sediment delivery from undisturbed land was 0.016 cubic yard per acre per year. Roads delivered 2.03 cubic yards per acre per year (127
times that of the undisturbed areas). Plantations, averaged across all ages, deliver 0.025 cubic yard per acre per year, or 1.56 times the undisturbed rate.

3. **Summer Cloudbursts:** Localized summer showers were shown to be a serious problem in burned granitic watersheds. Olsen Creek, a tributary of the Salmon River, delivered about 1,000 cubic yards to a debris catchment basin in response to a storm on September 25, 1990. This amounts to more than 1 cubic yard per acre of watershed.

### Effects of Floods, Landslide Episodes, and Mining on Channel Condition

1. **Floods:** The 1964 flood on the Salmon River caused major channel changes along the main river and most of the major tributaries. Changes included channel migration, aggradation, scour, and widespread loss of riparian vegetation. Most low-gradient floodplains were stripped of riparian vegetation and covered with fresh sediment. High-gradient reaches experienced both scour and aggradation. Loss of fish habitat is clearly apparent on aerial photographs. The floods of 1965–75 caused similar damage, but they were much less severe in most areas, and damage was more localized. The flood of 1955 was manifested by disruption of riparian vegetation on several tributary streams and by localized aggradation and scour. No major floods occurred between 1975 and 1990, allowing some recovery to occur.

2. **Mining Effects:** Hydraulic mining significantly altered the main river channels, changed the vegetation in the channel area (due in part to removal of soil down to bedrock), and delivered up to 15 million cubic yards of sediment into the river. One of the most disturbed areas was the upper South Fork of the Salmon River above its junction with the East Fork.

### Conclusions

**Potential for Future Landsliding in the Salmon River**

The major flood and landslide-producing storms during this century in the Salmon River Basin occurred from 1964 to 1974 when only 2.5 percent of the surface area of the watershed was disturbed by roads, harvest, or recent fire. Since that time, disturbance levels in the watershed have risen to 19 percent of the watershed. As a result, when storms such as those of the 1960’s and 1970’s reoccur, sediment production and its effects on the distribution and structure of riparian habitat, reasonably can be expected to greatly exceed that of the past.

**Future Fire Potential**

The potential for large intense fires in the future is very high due to the accumulation of large amounts of fuel over the past 50 years. Large fires occurred in 1917, 1918, 1977, and 1988. Such fires greatly increase the potential for landsliding and soil erosion.
Climatic Trends

Though future climatic trends cannot be forecast accurately, the historic record shows a pronounced drought in the mountains of northern California from 1915 to 1950, and the recent drought could persist for a similar length of time. Alternatively, we could enter a wet cycle similar to that of 1964–75. During wet periods, landslide episodes dominate the production of sediment, while during droughts, the majority of sediment is produced by surface erosion.

Current Riparian Habitat Condition Versus Historic Sediment Production

The correlation between past sediment production and current fish habitat conditions was poor; that is, watersheds that produced large volumes of sediment in the past 50 years did not necessarily exhibit poor riparian habitat when recent surveys were completed in 1990 and 1991. Some of the likely reasons for this lack of correlation are as follows:

1. It has been 28 years since the major channel-modifying event of 1964 and 18 years since the last large landslide-producing storm. Aerial photographs document the severe channel modifications over a large percentage of the channels in the watershed. There have been no large sediment-producing events since 1974, and channels which were filled with sediment and stripped of riparian vegetation have recovered to varying degrees. Studies of nearby watersheds (Liste 1981) similarly affected by the 1964 flood revealed that in confined channels, bed elevation returned to pre-flood levels within 15 years after the flood. This study identified considerable revegetation of damaged riparian areas by 1988.

2. Fish habitat surveys are very site-specific, and it is difficult to generalize these data for comparison to basin-wide sediment inventories. The habitat surveys were designed prior to this study and were not intended to be directly correlated in this way.

Recommendations

Due to increased landslide risk in the Salmon River watershed associated with present disturbance levels, the following measures are recommended:

Inventory and Mitigation of Existing Problems

1. Complete a basin-wide road management plan to identify sediment sources and eliminate unneeded road segments.

2. Prioritize and repair identified road problems. Emphasize repair of fills, critical erosion sites, degraded riparian areas, and landslides. Focus landslide stabilization on sites where the potential damage to habitat is high and there is a high likelihood that the landslide can be stabilized successfully.

3. A fire management strategy is needed for the watershed; the strategy should include re-introduction of fire into the ecosystem by prescribed burning, etc.
4. Continue planning watershed improvement projects. Make prioritized lists of critical sites for the watershed.

Prevention of Future Management-Associated Landslides and Erosion

1. Develop and implement standards and guidelines for managing the sensitive geomorphic terranes identified by this study. Certain terranes account for disproportionately large amounts of the total sediment delivered to the river system. Roads, timber harvest, and other soil- and vegetation-disturbing activities should be minimized on these lands. Geotechnical stabilization techniques should be applied where these lands are subjected to disturbance. The most sensitive of the terranes include the inner gorge, dormant landslide deposits, and dissected granitic terrane.

2. Conduct appropriate geologic, soil, and hydrologic investigations prior to conducting any significant soil- or vegetation-disturbing activities.

Further Enhance Our Understanding of Sediment and Fish Habitat

1. In selected stream reaches, conduct integrated inventories of fish habitat and channel conditions, including movement of sediment, while considering historic landslide and soil erosion episodes.

2. Digitize channel scour data collected by this study to allow quantitative assessment of the role of channel erosion in the sediment budget.

3. Assemble relevant physical and biologic data for the remainder of the Klamath River in a format similar to that used in this study to allow basin-wide analysis on a GIS system.

There are still many unanswered questions regarding the relationship between sediment production and fish habitat condition. The Salmon River study provides the essential background data for future focused studies to address movement of sediment through the stream system and the evolution of channel condition. It also provides background for monitoring the effects of future storms and management activities.

Land Use Planning

NEPA and NFMA provided the framework for resource planning from resource allocation (level I) to decisions on site stabilization (level III). Webster and Koler (1991) provided examples of cumulative watershed effects analysis for the Olympic National Forest at each of these levels.

The concept of integrated resource planning (figure 2E.1) involves resource allocation during land management planning (LMP) by implementation of projects, monitoring, and feedback to the LMP allocations.

During LMP (level I), a sediment-yield model, such as R1–R4 (Cline et al., 1981) or WATSED (1991), which generally is not calibrated for local conditions, may provide a first approximation of conditions within a watershed. Typical map scale is 1:63,360.
The 10-year schedule specifies which watersheds will be entered and how much disturbance may occur. At this level, managers need to know enough about the condition of each watershed to minimize cumulative effects and to document the need for rehabilitation of highly affected areas. Sediment-yield models can be used for this level, but sediment budget data provide a more complete understanding of geologic processes that may occur in response to natural and management-related events. Typical map scale is 1:24,000.

During scoping of projects (level II), a proposed action is evaluated in terms of the resources at risk and condition of the watershed to identify issues and opportunities. When sediment yields or budgets are expected to reach or exceed a threshold, a hazard-risk assessment may be required. This assessment includes soil slope movement, rock slope movement, and surficial erosional processes (e.g., rainsplash and sheetwash). The hazard-risk assessment may become useful during formulation of alternatives to concentrate or avoid activities on specific sites or to mitigate the hazards that put resources at risk. Typical map scale is 1:24,000 or 1:15,840.

Analysis of effects of each alternative includes estimating volumes of delivered sediment using sediment yield models or sediment budgets. Transport of the delivered sediment may be estimated if the channel geometry, gradient, material size, roughness, and 2-year and 100-year flows are known. Bed material fluxes can then be calculated to determine scouring or aggradation of the stream channel at stream segments of interest.

Following selection of a preferred alternative, a forecasting routine may be run to estimate when sediment levels might drop to the point that further activities in the analysis area can be planned. Optimum monitoring of the geomorphic and hydrologic processes within the watershed includes updating the landslide inventory, climate record, and channel changes (geometry, gradient, bedforms, and bed material size).

To calculate areas of scouring and aggradation effectively, bed material size distribution must be estimated to calibrate the flux of material into and out of the stream segment of interest. Field notes should include sketches and comments on changes in channel morphology. Koler (1992) estimated that these data can be collected in a couple of field days for a watershed approximately 8 square miles in area.

Monitoring data can be stored in a data base that includes a location on a GIS for quick retrieval.

Integrated resource planning (figure 2E.1) ends with feedback to the land management plan (LMP). Monitoring data and documentation supporting previous decisions may become relevant to LMP amendments.

Recommendations to Management

The challenge is not only to understand geomorphic and hydrologic processes, but to translate this understanding into a product that land managers can use to make sound decisions. The product should provide answers to such questions as what volume of material stored on the slope has the potential to be transported downslope into a stream, or the location of critical areas where sediment yield will be detrimental to resources such as fish habitat or municipal water supplies.
Managers need options for mitigation of increased sediment transport. In watersheds that exceed threshold values, it may be desirable to plan management activities while stabilizing sites to diminish sediment production and transport. Engineered structures may be feasible for slowing and stopping sediment (for the life of the structure, usually 25 to 50 years), or living "structures"—willow wattling bundles, for example—using biotechnology might be considered (usually with a life of several decades).

**Delineation of Unsuitable Lands**

Resource planning needs include delineating lands that fit the criteria for "unsuitable" for timber harvest due to irreversible soil loss as directed by NFMA (U.S. Congress, 1976). Delineation criteria have been established at regional and forest levels and may vary from place to place. For example, Pacific Northwest Region planning direction specifies a probability of increased rate or frequency of slope movement but does not identify the tools to be used for this determination, whereas forests in the Pacific Southwest Region have identified particular geomorphic features as unsuitable.

Delineating unsuitable acres has consequences for LMP allocations and allowable harvest. Monitoring plans can require periodic reporting of new areas delineated as unsuitable during site-specific projects.


References


**Stability Analysis for Road Access, Version 1.0 (SARA 1.0).** In process. Moscow, ID: U.S. Department of Agriculture, Forest Service, Intermountain Research Station, Engineering Technology.


References


APPENDIX 2.1
This basin study checklist is intended to provide an organizational framework for completing slope stability investigations and analyses at level I (drainage basin), level II (project), and level III (site-specific) scales.

Develop a logical sequence, or checklist, for accomplishing an analysis. The checklist is not intended to duplicate other information or methods already covered but as a cross-reference to all sections in the guide.

Courtenay Cloyd, Forest Engineering Geologist, Siuslaw National Forest

A level I analysis is a relative landslide hazard evaluation for resource allocation (Prellwitz et al., 1983).

1. Project Objectives (What are we going to do?)
   - See sections 1B, 1C.1, 1C.2, 1C.4, and 2.
   - Identify those parts of an analysis area having a high probability of slope failure under natural conditions.
   - Assess the extent to which roads and timber harvest will affect or be affected by the existing slope stability of the area.
   - Determine the geologic materials and processes occurring in analysis areas as elements of cumulative effects assessments.
   - Identify private property and facilities and public resources that might be at risk if landslides were to occur.
   - Rule of Thumb: Work closely with those requesting the slope stability assessment to identify analysis area boundaries, as well as resources, property, and facilities potentially at risk.

2. Initial Literature Review (Past History)
   - See sections 1B, 1C.4, and 2.
   - Review:
     (1) Published and in-house geologic maps and reports.
(2) Existing landslide inventories.

(3) Forest Soil Resource Inventory (SRI) or other soils mapping.

(4) Rainfall/climate models.

(5) Seismic hazard ratings.

- An overview of 1:40,000 and 1:24,000 aerial photography can be performed at this time.

- Rules of Thumb:

  (1) Look for patterns of failure related to landforms: slope angle, aspect, position, or lithology; structure; or soil type.

  (2) Look for relationships between the type of management activity and slope failure.

  (3) Look for resources potentially at risk not previously identified.

3. Field Reconnaissance (Current Condition)

- See sections 1C.4 and 2.

- Check interpretations made in preliminary aerial photography review.

- Check mapped soils and geology against field units.

- Rules of Thumb:

  (1) Tree and vegetation cover often mask existing failures.

  (2) Hydrophilic and dry-site vegetation may help characterize basic ground water/soil moisture conditions.

  (3) Hummocky slope surfaces, and/or pistol-butted, bent, bowed, or leaning trees may indicate slow, relatively recent slope movement where other evidence is absent. However, pistol-butted trees may also be the result of snow loads.

4. Data Synthesis and Initial Hypothesis

- See sections 1B.2 and 2 and the LISA 2.0 User's Manual (Hammond et al., 1992).

- Interpret aerial photography.

  (1) Identify landforms.

  (2) Determine slope failure modes: shallow (planar/translational) or deep-seated (circular/ rotational).
(3) Determine rates of slope movement (slow/rapid).

(4) Determine where failed materials are deposited.

- Create overlays of rock and soil types and mapped structural features at 1:24,000 scale.
- Develop initial model of the processes and rates, and the causes and effects, of slope movement occurring in the analysis area.
- Identify polygons for LISA based on the information gathered above.
- Develop a plan to verify the model in the field.

- Rules of Thumb:
  (1) Use one or more sets of early aerial photographs, together with climate models and management history, to gain perspective on process-and-rate, cause-and-effect questions.
  (2) The GIS can be an important tool for data analysis and landform identification.

5. Field Exploration (Test Hypothesis)

- See sections 1C.4 and 3.

- Gather information about typical soils, landforms, and ground water characteristics in proposed LISA analysis polygons or areas considered to have a high likelihood of slope failure based on qualitative assessment and professional judgment.

- Confirm/modify landslide inventory information from aerial photography interpretation.

- Measure field-developed cross-sections of typical landforms.

- Determine typical soil depths and ground water conditions using a soil auger or drive probe. Locate springs and seeps.

- Sample soil units to verify classification. Use USC field test methods or laboratory analyses.

- Rule of Thumb: Field exploration emphasis is on verifying available published and office information and photo interpretations by visiting representative areas. No attempt is made to field-check major portions of the project area at this level of analysis.
6. Analysis

- See sections 1C.4, 2, and LISA 2.0 User’s Manual.

- Empirical

  (1) Combine office and field information to define areas where landslides have a high, moderate, or low likelihood of occurring under natural and managed conditions.

  (2) Ratings are based on knowledge of local geology, soils, and hydrology and on professional judgment.

- Probabilistic or Rational

  Probabilistic (LISA): evaluate the relative landslide hazard of an analysis area from the distribution and variability of soil strength parameters and ground water based on measured or assumed values.

- Rules of Thumb:

  (1) Both empirical and rational analysis methods will provide an estimate of landslide HAZARD: the probability of slope failure.

  (2) RISK is an assessment of the socioeconomic consequences of slope failure.

  (3) Responsible land management decision making is based on consideration of both the hazard and risk of slope failure.

7. Report and Recommendations

- See section 1C.4 and the LISA 2.0 User’s Manual.

- Summarize the results of investigations and analyses according to standard professional geologic and engineering geologic report formats.

- Rule of Thumb: Integrating analysis results into environmental analysis (NEPA) and planning processes will require close, frequent interaction with everyone on those teams. Written reports are seldom adequate to explain hazard and risk assessments to those unfamiliar with the concepts.

Level II

Michael Long, Forest Engineering Geologist, Willamette National Forest

Introduction

A level II analysis is used for project planning; for example, a timber sale with two or three individual units and the transportation system consisting of one to three roads (section 1C.2).
1. Project Objectives (What are we going to do?)

- Accomplish presale reconnaissance and transportation planning to identify areas of questionable stability either for level III analysis or to avoid them by altering the transportation route and/or harvest plan (section 1C.4).

- Rule of Thumb: Work closely with presale and transportation planners to understand their objectives. Discuss their project constraints to determine how flexible the road locations and sale unit boundaries are (due to budget, multi-use transportation route, and so forth).

2. Initial Literature and Document Review (Past History)

- See sections 1C.4 and 2.

- Obtain 1:24,000 or 1:12,000 aerial photographs.

- Obtain project topographic maps at 1:6,000 (1 inch = 500 feet) or 1:3,600 (1 inch = 300 feet) scale.

- Review any previous geotechnical or geologic reports done in the area.

- Review the Forest Soil Resource Inventory (SRI) for pertinent data.

- Review climate and rainfall data.

- Rules of Thumb:

  1. Make at least three copies of topographic maps (one for the field, one for data entry, and one for the report).

  2. Take SRI data to the field and verify—share results with watershed group.

3. Field Reconnaissance (What is current condition?)

- See sections 1C.4, 3B.1, 3B.2, 3C, and 3D.

- For transportation routes, this is commonly referred to as a flag or tag line (prior to survey control) or P-line investigation (after survey control has been established).

- For timber sale units, this consists of a reconnaissance of the entire unit on the ground.

- As the route or sale unit is investigated, the area may be divided into sections or cells defined by topography, soil classification, or design criteria.

- Obvious and incipient features of stability concern may be assigned individual distinctive cells.

- Field-developed cross-sections are measured to typify each area.
Soil and rock units are established and identified. Small samples may be obtained for laboratory analysis for clay content and gradation.

Rules of Thumb:

1. Take plastic bags to the field for samples and to protect aerial photographs and maps.

2. Map transportation routes by “design segments” (full cut, cut and fill, through fill).

3. Take soil auger, drive probe, and small shovel to obtain samples and to determine soil depth and ground water conditions.

4. Use geophysics (seismic and resistivity methods) when necessary.

4. Data Synthesis and Initial Hypothesis

- See sections 1C.4, 2, 4B, 4E, 5B, and 5C.

- Prepare field-developed cross-sections in stability analysis format shown in plan and section.

- Estimate soil and rock shear strength parameters and ground water conditions.

- Calculate factors of safety according to road design template using SARA (Stability Analysis for Road Access, in process) or timber sale unit areas using DLISA (Deterministic Level I Stability Analysis, 1991).

- Develop level III investigation plan if necessary.

- Rule of Thumb: Use a sensitivity analysis approach to refine shear strength parameters and ground water model.

Level III

Mark Leverton, Engineering Geologist, Willamette National Forest

Introduction

A level III analysis is site-specific, measured in hundreds of square feet to a few acres. Mapping is usually generated by the investigator at a scale ranging from 1" = 10' to 1" = 100'.

Suggested Checklist

1. Project Objectives

   - Assess current stability.

   - Assess impacts due to management activities.

   - Provide range of feasible alternatives and costs.

   - Assess risks associated with each alternative.
• Rules of Thumb:
  (1) Determine sideboards (cost, management restrictions, etc.).
  (2) Stay in contact with pertinent specialists.
  (3) Keep intensity of investigation consistent with risk.

2. **Initial Literature and Document Review (Past History)**

• More intensive toward mechanics of failure.

• Local geomorphology.

• Soil and rock shear strength.

• Previous local investigations.

• Rules of Thumb:
  (1) Any information is better than none.
  (2) Discuss with people acquainted with the area.
  (3) Determine and reestablish past survey control points.

3. **Field Reconnaissance (What is current condition?)**

• See section 3C.

• Have field-developed cross-sections been measured?

• Have soil and rock units been established?

• Consider and design exploration plan.

• Rules of Thumb:
  (1) Establish reproducible survey control.
  (2) Take thorough notes understandable to someone else.
  (3) End each field day with review of your work and needs.
  (4) Photograph pertinent features.

4. **Data Synthesis and Initial Hypothesis**

• First: approximate subsurface interpretation and mechanics of failure.

• Last: develop exploration plan and budget.
• Rules of Thumb:
  
  (1) Do not be afraid of documenting your initial interpretation.
  
  (2) Have multiple hypotheses for subsurface conditions.
  
  (3) Determine what is needed to prove your model.
  
  (4) Determine what needs to be monitored and where.
  
  (5) Complete drawings and interpretation before exploration.

5. Field Exploration (Test Hypothesis)

• See section 3D.

• Drive probes/geophysics/backhoe/drill/dye testing/etc./field material tests.

• Instrumentation/field logging methods.

• Rules of Thumb:
  
  (1) Dress appropriately.
  
  (2) Work safely. This is particularly important at the end of the day.
  
  (3) Anticipate equipment needs and contingencies.
  
  (4) Modify and verify interpretation on drawings serially.
  
  (5) Expect and integrate subsurface “surprises.”
  
  (6) Monitor ground water encountered for changes.
  
  (7) Modify exploration plan as needed.

6. Laboratory Testing

• See section 4C.

7. Office Data Development

• Take core photographs and complete final logs.

• Develop final subsurface model.

• Rules of Thumb:
  
  (1) Leave a paper trail of observations and assumptions.
  
  (2) Document what is known AND what is unknown.
8. Analysis/Develop Alternatives

- XSTABL (1992)
- GW (Prellwitz, 1990)
- DLISA (1991)
- Document analysis assumptions.
- Rules of Thumb:
  1. Handwritten notes on computer printouts are good.
  2. Are assumptions consistent with field observations?

9. Report/Recommendations

- Clear, thorough, brief report (see first rule of thumb below).
- Present the full range of feasible alternatives.
- Present risk assessment and cost/benefit analysis.
- Rules of Thumb:
  1. Consider writing a thorough report for your files with procedure, assumptions, knowns, unknowns, classifications, analysis, etc., and a brief, clear memo (one or two pages) documenting results, alternatives, costs, risks, and recommendation for your client.
  2. Treat the documentation process as though someone else will be continuing the work you started. Be thorough.

10. Construction

- See section 6.
- Rules of Thumb:
  1. If you designed it, be there when it is built.
  2. Verify assumptions during excavation.
  3. Modify interpretive drawings if necessary.

11. Post-Construction Monitoring

- See section 6K.
- Rules of Thumb:
  1. The monitoring system should be functional for several years.
  2. Ensure reproducible survey control.
Department of Agriculture, Forest Service, Intermountain Research Station, Engineering Technology.


Stability Analysis for Road Access, Version 1.0 (SARA 1.0). In process. Moscow, ID: U.S. Department of Agriculture, Forest Service, Intermountain Research Station, Engineering Technology.

2.2 LISA Problem Example

This example problem is reprinted from *Level I Stability Analysis (LISA) documentation for Version 2.0* by C. Hammond, D. Hall, S. Miller, and P. Swetik, General Technical Report INT-285, U.S. Forest Service, Ogden, UT.

There appears to be an error in Table 6.2. The probability of failure ($P_f$) for polygon 5D should be about 0.08–0.09, for a moderate hazard rating.
CHAPTER 6 — EXAMPLE APPLICATION:
DARK 3 PLANNING AREA, GIFFORD
PINCHOT NATIONAL FOREST

6.1 Introduction

The Dark 3 planning area is on the Randle Ranger District, Gifford Pinchot National Forest, in Washington (Pacific Northwest Region). Jones (1990) performed a Level I stability analysis over the entire area and evaluated three timber sale alternatives. Figure 6.1 shows the topographic map of the Dark 3 planning area and the Level I polygons. (Unlabeled polygons are primarily flood plain deposits and were not analyzed.) The District then requested additional analysis (Level II) on one harvest unit for which field observations supported by the initial LISA analysis indicated a high probability of failure after timber harvest. The District desired to harvest the potentially unstable unit for silvicultural reasons. Both analyses will be described in this chapter. (Using LISA to perform a Level II analysis is discussed in section 5.1.3.)

6.2 Geology, Soils, and Topography

The bedrock geology and soil conditions of the Dark 3 planning area are shown in the Geologic Resources and Conditions (GRC) map (fig. 6.2). The bedrock geology of the western half of the area consists of extrusive igneous and minor pyroclastic rocks dipping to the west at 5 to 15 degrees. This bedrock forms a tablelike topographic surface with surface slopes ranging primarily from 20 to 50 percent. The overlying soils consist of 2 to 5 (locally 10+) feet of colluvium and residuum (GW-SM) with minor amounts of glacial till (SM-GM and GMu). It was anticipated that this region would have few stability problems because of the gentle slopes and therefore was analyzed with only two LISA polygons (designated as 3M and 4W in fig. 6.1).

A crescent-shaped area of steep ground with slopes generally greater than 70 percent extends from the northwestern to the southeastern boundary of the planning area. The soils of this steep crescent, which is the edge of the table of volcanic rocks, generally consist of 1 to 2 feet of coarse tephra overlying 2 to 3 feet of colluvium and minor residuum developed from the underlying volcanics (SM-GW). While most of the area appears dry and well drained, areas of springs and seeps are observed. The elevated groundwater and steep slopes apparently have caused rockfalls and debris avalanches, several of which are mapped on the GRC map. Because of the steep slopes and past failure activity, there was concern that timber harvest or road construction in the area would increase the mass failure potential with the possible impacts of loss of the soil resource and damage to the water quality and fisheries of Summit Prairie Creek. Therefore, the crescent was divided into several small polygons of four types (1D, 1M, 2D, 2M), differentiated by slope and groundwater conditions.

In the northeastern third of the area, the bedrock consists of pyroclastic rocks with minor intrusive and extrusive igneous rocks. The bedrock is overlain by glacial till, colluvium, and residuum with minor alluvium, averaging 5 to 10 (locally 30+) feet in thickness. The topography consists of moderate slopes (40 to 90 percent). The area is generally considered to be dry with low failure potential and therefore was analyzed with one LISA polygon (5D).
Figure 6.1—Dark 3 planning area and Level I polygons.
6.3 Polygon Delineation and Distribution Selection—Level I

Jones (1990) delineated polygons for the initial Level I analysis using 1:7,200 topographic maps and the soil/geology type as mapped in the GRC. In areas with slopes greater than 65 percent, additional polygons were delineated using low-altitude aerial photographs to better describe slope and groundwater characteristics. Initial soil type and soil depth estimates were obtained from the GRC map and the Soil Resource Inventory (SRI). Shear strength and unit weight values and distributions then were estimated from the USC classification and previous experience and by using table 5.1 and figure 5.11 of this manual. Groundwater distributions used were developed from the groundwater characteristics mapped on the GRC, field observations, and by using a catalog of distributions tied to various landforms developed by Wooten (1988). Root strength distributions used were those suggested by Wooten (1988) for a type B soil-root morphology class. Figure 6.3 contains Wooten's suggested distributions. Limited field checking was performed to verify office findings. Table 6.1 gives the distributions used in the analysis.

6.4 Level I Results

Table 6.2 lists the ranges of the probabilities of failure for each polygon as estimated using the LISA program for both the natural and clearcut states. The range of probability of failure values was obtained from five simulations, each using a different seed number for the random number generator. The probabilities of failure for clearcut harvest are conditional on a "major" rainfall or rain-on-snow event occurring during the period of minimum root strength. Also given are relative probabilities of landslide hazard based on the experience and interpretation of the Gifford Pinchot National Forest geotechnical group. This scale can aid individuals not familiar with the LISA program and those uncomfortable with probability numbers in interpreting LISA results. It is not an absolute scale that would necessarily be applicable elsewhere; it is only a relative scale based on the experience of the geotechnical group on the Gifford Pinchot.

The proposed cutting units for three timber sale alternatives were overlain on the LISA polygons, and the land area in low, moderate, and high failure-potential polygons was measured. These results are summarized in table 6.3. For each proposed cutting unit, the potential impacts should a failure occur were evaluated as either localized or as having the potential to deliver sediment to Summit Prairie Creek. One of the harvest units (unit 7) of timber sale alternative 1 was located partially in the high failure-potential polygon 2M, with the potential impact of delivering sediment to the creek. Because of this LISA result, along with observations of instability along road 2325 above unit 7, further analysis of the unit was deemed necessary. This analysis is discussed in the next two sections.

6.5 Polygon Delineation and Distribution Selection—Level II

Jones (1990) spent approximately 3.5 days in the field gathering slope, soil type, soil depth, and groundwater information to further evaluate the portion of the Dark 3 planning unit surrounding harvest unit 7. Based on the field evaluation, Jones modified the polygons in that portion as shown in figure 6.4. Slopes were measured with a clinometer and soil depth with a hand auger at random...
Figure 6.2—Geologic Resources and Conditions (GRC) map for the Dark 3 planning area.
NAME/ORIGIN: Colluvium, residuum, and local deposits of glacial till underlying pyroclastic and minor intrusive and extrusive igneous rock.


ROCK: Tuff, tuff breccia, subordinate felsic tuff (URC: BCEE–DDEC); basalt, andesite (URC: BBBEA), BRU 40050D.

SIGNIFICANT CONDITIONS: Plastic soil is not free-draining, susceptible to slope failure when disturbed on steep slopes, and generally a weak subgrade material. There is a low potential for material sources in this unit. Refer to map unit ☐ for significant conditions for glacial till.

NAME/ORIGIN: Glacial till, colluvium, residuum, and minor alluvium underlying pyroclastic and minor intrusive and extrusive igneous rock.


ROCK: Tuff, tuff breccia, local felsic tuff (URC: BCEE–DDEC); basalt, andesite (URC: BBBEA), BRU 40050D.

SIGNIFICANT CONDITIONS: Till is locally plastic and/or compact and not free-draining resulting in elevated water tables. Loose till is subject to raveling resulting in increased road maintenance. Plastic soil is susceptible to slope failure when disturbed on steep slopes, and is generally a weak subgrade material.

NAME/ORIGIN: Colluvium, residuum, and local deposits of glacial till underlying extrusive igneous and minor pyroclastic rock.


ROCK: Basalt (URC: BBEA); andesite (URC: BBEA–DDEC); basalt breccia, tuff, tuff breccia (URC: BCEE–CCEB), BRU 2021.

SIGNIFICANT CONDITIONS: Unit is characterized by gentle to moderate slopes with poor surficial drainage indicated by the presence of wet areas. Plastic residuum is not free-draining and is generally a weak subgrade material. Glacial till—refer to significant conditions of map unit ☐. Till occurs mainly in the Dark Creek drainages.

Special Considerations
- The compartment is overlain with 2–4' of past and recent Mount St. Helens pumice and ash consisting of poorly graded sand to silty sand (USC:SP–SM). Tephra is free-draining, easily eroded, and may be washed and accumulated into thicknesses up to 12'.
- Several sidecast failures occur along the 29 Rd. adjacent to McCoy Creek.

Figure 6.2—(Con.)
Figure 6.3—Root cohesion distributions suggested by Wooten (1988) for use on the Gifford Pinchot National Forest
Table 6.1—Distributions used in the Dark 3 Level I analysis

<table>
<thead>
<tr>
<th>Polygon</th>
<th>$D$</th>
<th>$\alpha$</th>
<th>$\gamma$</th>
<th>$\phi'$</th>
<th>Natural</th>
<th>Clearcut</th>
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</thead>
<tbody>
<tr>
<td>1D</td>
<td>$[1, 3, 5]$</td>
<td>$T[65, 75, 95]$</td>
<td>$U[10, 75]$</td>
<td>$U[31, 38]$</td>
<td>$T[0, 2.4]$</td>
<td>$T[0, 25.5]$</td>
</tr>
<tr>
<td>1M</td>
<td>$[1, 3, 5]$</td>
<td>$T[65, 75, 95]$</td>
<td>$U[10, 75]$</td>
<td>$U[31, 38]$</td>
<td>$T[0, 2.5]$</td>
<td>$T[0, 3.6]$</td>
</tr>
<tr>
<td>2D</td>
<td>$[1, 3, 5]$</td>
<td>$T[70, 85, 110]$</td>
<td>$U[10, 75]$</td>
<td>$U[31, 38]$</td>
<td>$T[0, 2.4]$</td>
<td>$T[0, 25.5]$</td>
</tr>
<tr>
<td>2M</td>
<td>$[1, 3, 5]$</td>
<td>$T[70, 85, 110]$</td>
<td>$U[10, 75]$</td>
<td>$U[31, 38]$</td>
<td>$T[0, 2.5]$</td>
<td>$T[0, 3.6]$</td>
</tr>
</tbody>
</table>

For all polygons, $\gamma$:

- $C_r$ (Natural): $H[4, 5, 80, 10, 5]$ (or $H[7.5, 20, 20, 20, 10, 5]$, see fig. 6.3)
- $C_r$ (Clearcut): $H[4, 5, 40, 45, 10]$ (see fig. 6.3)

$\gamma_r$:

- $w$: $U[10, 25]$

$G_r$:

- 2.4

Table 6.2—Dark 3 Level I results

<table>
<thead>
<tr>
<th>Polygon</th>
<th>Natural state</th>
<th>Hazard$^1$</th>
<th>Clearcut state</th>
<th>Hazard$^1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$P_f$</td>
<td>VL</td>
<td>$P_f$</td>
<td>L to M</td>
</tr>
<tr>
<td>1D</td>
<td>0.005-0.010</td>
<td>VL</td>
<td>0.073-0.085</td>
<td>L to M</td>
</tr>
<tr>
<td>1M</td>
<td>0.008-0.013</td>
<td>VL</td>
<td>0.091-.119</td>
<td>M</td>
</tr>
<tr>
<td>2D</td>
<td>0.025-0.040</td>
<td>VL to L</td>
<td>0.161-.174</td>
<td>H</td>
</tr>
<tr>
<td>2M</td>
<td>0.029-.043</td>
<td>VL to L</td>
<td>0.201-.223</td>
<td>H</td>
</tr>
<tr>
<td>3M</td>
<td>0.000-.000</td>
<td>VL</td>
<td>0.000-.002</td>
<td>VL</td>
</tr>
<tr>
<td>4W</td>
<td>0.000-.000</td>
<td>VL</td>
<td>0.000-.002</td>
<td>VL</td>
</tr>
<tr>
<td>5D</td>
<td>0.014-.024</td>
<td>VL</td>
<td>0.176-.215</td>
<td>H</td>
</tr>
</tbody>
</table>

$^1$Relative hazard based on experience of Gifford Pinchot National Forest geotechnical group:

- 0-0.029 = Very low (VL)
- 0.030-0.079 = Low (L)
- 0.080-0.159 = Moderate (M)
- 0.160-0.249 = High (H)
- 0.250+ = Very high (VH)

Table 6.3—Summary of potentially unstable slopes affected by timber harvest

<table>
<thead>
<tr>
<th>Acres affected</th>
<th>Alt. 1</th>
<th>Alt. 2</th>
<th>Alt. 3</th>
</tr>
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<tbody>
<tr>
<td>Low hazard</td>
<td>16.3</td>
<td>13.7</td>
<td>11.0</td>
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<tr>
<td>Moderate hazard</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>High hazard</td>
<td>4.5</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
locations when a change in conditions was perceived. Jones recognized that depth measured to "refusal" using a hand auger may not necessarily be the depth to bedrock, as cobbles and boulders also can cause refusal. Therefore, the maximum depth used in the input distributions was somewhat greater than actually measured in the field. The soil type was finer textured (SP-SM) than was predicted by the GRC maps, with slightly plastic fines, and was easily excavated by hand ($D_s$ of 25 to 45 percent). Shear strength and unit weight values for this different soil type were again estimated from Table 5.4 and Figure 5.11 of this manual. Several springs were observed in areas that were assumed to be dry in the Level I analysis, although the slopes were relatively dry overall. Therefore, distributions were developed to describe the observed conditions, rather than using the catalog of distributions developed by Wooten (1988). The distributions used for each polygon are given in Table 6.4.

### 6.6 Level II Results

Table 6.5 gives the probabilities of failure and relative landslide hazard for each polygon. The more detailed Level II analysis using the LISA program indicates that a large portion of harvest unit 7 has a very low to low probability of failure even after timber harvest, primarily because of the gentle slopes. However, approximately 4.7 acres lie in moderate landslide hazard ground with localized failure impact, and 3.9 acres lie in high landslide hazard ground with a high likelihood of sediment entering Summit Prairie Creek should a failure occur.

Based on the Level II analysis, the District modified the unit boundary to omit the 3.9 acres having high landslide hazard. In addition, because of the observed indications of instability on the fill slope of road 2325 through the unit, Jones (1990) recommended that if timber sale alternative 1 was selected as the preferred alternative, further Level II analysis using the SARA program should be performed on the existing road and on any proposed new construction in harvest unit 7 to determine the need for further subsurface investigation, and stability analysis and design (Level III).
Figure 6.4—Level II analysis polygons.
### Table 6.4—Distributions used in the Dark 3 Level II analysis

<table>
<thead>
<tr>
<th>Polygon</th>
<th>$D$</th>
<th>$\alpha$</th>
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<tbody>
<tr>
<td>12M</td>
<td>$T[2.35,5]$</td>
<td>$T[60,70,85]$</td>
</tr>
<tr>
<td>32M</td>
<td>$T[2.35,5]$</td>
<td>$T[65,75,95]$</td>
</tr>
<tr>
<td>52M</td>
<td>$T[2,4,6]$</td>
<td>$T[35,55,65]$</td>
</tr>
</tbody>
</table>

For all polygons, $\theta$: $U[6,12]$

$C_r$ (Natural): $H[4.5, 80, 10.5]$ (see fig. 6.3)

$C_r$ (Clearcut): $U[4.5, 40, 45, 10]$

$\gamma_d$: $N[95, 5]$

$\omega$: $U[10, 25]$

$G_z$: 2.4

$C_r'$: $U[20, 75]$

$\phi$: $B[28, 36, 2, 2]$

$D_u/D$ (Natural): Histogram Min Max %

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0.1</td>
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<td>1</td>
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<td>1</td>
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<tr>
<td>.9</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

$D_u/D$ (Clearcut): Histogram Min Max %

<p>| | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
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<tbody>
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<td>3</td>
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<tr>
<td>.4</td>
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<td>.5</td>
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<td>6</td>
</tr>
<tr>
<td>.6</td>
<td>.7</td>
<td>1</td>
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<tr>
<td>.7</td>
<td>.8</td>
<td>1</td>
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<td>.8</td>
<td>.9</td>
<td>1</td>
</tr>
<tr>
<td>.9</td>
<td>1</td>
<td>1</td>
</tr>
</tbody>
</table>

### Table 6.5—Dark 3 Level II results

<table>
<thead>
<tr>
<th>Polygon</th>
<th>$P_f$</th>
<th>Hazard</th>
<th>$P_f$</th>
<th>Hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>12M</td>
<td>0.019-0.025</td>
<td>VL</td>
<td>0.117-0.125</td>
<td>M</td>
</tr>
<tr>
<td>32M</td>
<td>0.026-.039</td>
<td>VL to L</td>
<td>.210-.244</td>
<td>H</td>
</tr>
<tr>
<td>52M</td>
<td>0.001-.004</td>
<td>VL</td>
<td>0.009-.014</td>
<td>VL</td>
</tr>
</tbody>
</table>

1. Relative hazard based on experience of Gifford Pinchot National Forest geotechnical group:

0-0.029 = Very low (VL)

0.029-0.079 = Low (L)

0.079-0.159 = Moderate (M)

0.159-0.249 = High (H)

0.249+ = Very high (VH)
References


2.3 Geologic Information Management

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THOMAS K. REILLY
U.S. Forest Service

INTRODUCTION

The Management Situation

The geologic history, origin, and processes of a forest landscape have fundamental influences on the capabilities and limitations of the land. The geologic characteristics of an area influence landform formation, soil profile development and soil characteristics, and ground-water occurrence; they consequently influence vegetative types and patterns. Areal geology affects the ability to manage a forest without causing adverse environmental effects or impairing future land productivity. The geology of an area also provides opportunities for recreation, education, and scientific research. Knowledge of the geologic capabilities and limitations of a national forest should result in a better understanding of the trade-offs and "opportunity cost" of alternative management strategies.

Comprehensive geologic information is not easily available to planners at most national forests. This is because the U.S. Forest Service lacks a uniform system to store and display basic geologic information. A system does not exist for a variety of reasons, including a lack of a clearly demonstrated need, overestimation of the costs of acquiring such information, and the misconception that the information is already available from other resource inventories.

Recognition of the Need for a Geologic Information System

Several problems that occurred on the Gifford Pinchot National Forest (GPNF) underscored the need for a geologic information system.

- Availability of information was people-dependent. It was recognized that information became lost, stored in project files, or worse, stored in people's minds. If employees left the GPNF, they took with them information gathered at considerable expense.
- The quality of information was varied. With no consistent terminology, decisions about how to compile, save, and apply information were subjective and depended on the level of experience of the individuals involved.
- Involvement was often late and costly. Geologists working on the GPNF are assigned to the Engineering Branch, due to the traditional role of geologists in project design. Because of the lack of a geologic information system and time to gather planning-level information on a project basis, geologic expertise was usually sought only after planning decisions had been made. Many decisions involved road building or timber harvest in an area where, because needed geologic resources were lacking or the geologic conditions were adverse, difficulties were later encountered.
- Involvement in planning was recognized as being valuable. If basic geologic information was available at even a small scale, the consistencies (or inconsistencies) of geologic resources, conditions, and processes might be projected across an area between locations for which some information was available. If such projected information was readily available, planners and land managers would become aware that geologists could be involved in planning efforts on a cost-effective basis. Informed decisions pay off in reduced costs and improved resource protection.

While the above factors were recognized by geologists, it was not until late 1979 that the GPNF Forest Management Team became aware of the usefulness of a geologic information system. The Forest Planning Interdisciplinary Team was then being formed to develop a Land and Resource Management Plan in compliance with the National Forest Management Act. The Forest Geologist was selected to be a team member. A proposal for formulation of a system to support the planning effort was soon accepted. The intent of the proposal was not only to comply with the needs of the GPNF planning effort, but also to break out of the project mode and use the system in areal analysis. It was predicted that if a well-planned, long-term comprehensive geologic information system was developed, it would result in better management decisions based on better information.
Development of a System

Careful thought was given to format, structure, classification systems, and terminology for the new system. Considerable effort was required to ensure that the system would be technically valid and internally consistent, use nontechnical terminology, and be easily used by geologists and non-geologists alike. Also, organizational commitment was gained to ensure that the system would be technically valid and improved through regular updating. Coordination was required to ensure that duplication and overlap with other resource systems (such as soils, hydrology, and minerals) did not occur. The system was tailored to satisfy management needs not already being met.

This paper reviews the geologic information system and some actual and potential general applications. The final sections describe some examples of system information use.

OVERVIEW OF THE GEOLOGIC RESOURCES AND CONDITIONS SUBSYSTEM

The initial geologic information system was developed after formulating standards and guidelines for information collection. In response to user needs, the format evolved to the present configuration. At the urging of forest managers, the system was included as one component of an overall comprehensive multi-resource information system, and it was designated the Geologic Resources and Conditions (GRC) subsystem. The overall system, known as the Total Resource Information (TRI) system, divides the entire GPNF into more than 200 distinct geographic areas (referred to as compartments) averaging about 6,000 acres each.

The name Geologic Resources and Conditions recognizes the basic division of geologic factors into resource development opportunities and potential constraints to certain land-management activities. The map scale used is 4 in. = 1 mi, which matches that of the larger TRI system. A user Subsystem Guide (U.S. Department of Agriculture, Forest Service, 1986b [hereafter U.S. Forest Service]) describes the subsystem, terms, systems, and methods used and possible applications of the GRC subsystem information. Federal regulations, derived from Acts of Congress, and Forest Service manual descriptions derived from those regulations refer to geologic factors as opportunities and constraints to land-management activities. Geologic opportunities include potential development and use of geologic resources. Geologic constraints are factors which affect cost, safety, or feasibility of management activities.

The GRC map system is designed to display basic geologic information in a simple, flexible format. Any land manager can have such a system developed at no major cost, if data and information gathered for site-specific project work are routinely plotted on system map overlays. GRC maps were developed in a three-step process.

- Office preparation: All available information was compiled on the 4 in. = 1 mi system base map overlays. Sources of information included project files and reports, published geologic information, and results of air photo interpretation.
- Field verification: Whenever possible, this step was accomplished in conjunction with other priority fieldwork. On the whole, only a slight increase in field time was required for verification, which greatly enhanced the product.
- Compilation: Boundaries were reconciled, the final map prepared, and a folder developed to store all available maps, photos, and records.

The Forest Service has established a national Geographic Information Systems (GIS) steering committee with a mandate to monitor, evaluate, and recommend ways to implement technologies and develop a national GIS strategy. Looking ahead to future GIS technology, GRC maps were developed on a controlled (spatial) base map referenced to geographic coordinates to allow accurate digitizing of geologic material unit boundaries and the location of landforms and geologic resources. The GRC subsystem is fully compatible with GIS concepts and software and can be readily digitized.

DETAILED SUBSYSTEM DESCRIPTION

Philosophy of the Subsystem

The GRC subsystem is user-friendly by design: it is a straightforward, single-sheet product that uses clear symbols and nontechnical terms. It is intended for general understanding by planners, resource specialists, and land managers.

GRC Subsystem Components

Components of the GRC subsystem are the map, the Legend, Materials Sources Information, and Special Considerations information block. Figure 1 is a representation of a GRC map.

All GRC maps are drawn on 24 in. x 36 in. reproducible mylar sheets. Transparent adhesive-backed pattern film is used to distinguish the various geologic materials units on the map sheet and on the left margin of the descriptive block. The user can locate the descriptive information in the information block keyed by the same film pattern.

A simplified representation of a GRC map sheet is given in Figure 2. On an actual map, the local road network is shown to help the user locate a feature or project. Annotations surrounding the map describe various features typical of GRC maps. If new information is obtained that results in a changed interpretation of materials unit boundaries, the map can easily be changed. The pattern film on the master copy GRC map
Figure 1. Sample Geologic Resources and Conditions (CRC) map layout.

Figure 2. Example of a GRC topographic base map.
is removed, the boundary redrawn, new pattern film attached, and a new (updated) mylar copy is made from the master copy.

An enlargement of a typical geologic materials unit descriptions is shown in Figure 3. The left portion of the description displays the same pattern film as those materials units shown on the base map. Geologic conditions that may affect current or planned land management are outlined under "Significant Conditions".

<table>
<thead>
<tr>
<th>NAME/ORIGIN</th>
<th>GENERAL DESCRIPTION OF UNIT</th>
</tr>
</thead>
<tbody>
<tr>
<td>SOIL</td>
<td>ORIGIN, TEXTURE, CLASSIFICATION BY ENGINEERING CHARACTERISTICS</td>
</tr>
<tr>
<td>ROCK</td>
<td>ORIGIN, TEXTURE, CLASSIFICATION BY ENGINEERING CHARACTERISTICS</td>
</tr>
<tr>
<td>SIGNIFICANT CONDITIONS</td>
<td>SUMMARY OF GENERAL GEOLOGIC CONDITIONS WITHIN DESCRIBED MAP UNIT</td>
</tr>
</tbody>
</table>

Figure 3. Typical Geologic Materials Unit (GMU) descriptions.

Figure 4 is the Materials Source Information for a typical GRC map. Each known usable source of construction materials is located on the GRC map by symbol and described in the map margin information block shown. The symbol on the map indicates the status (for example, developed or closed). If the information block is left blank, the absence of a known source is pointed out in the Special Considerations information block.

Some other types of information blocks are not shown in the figures. These are:

1. GRC map Special Considerations: This space is used to summarize the significant geologic resources and conditions for the entire TRI compartment.

2. GRC map legend: The legend displays and describes the symbols that locate geologic resources and conditions and identify geologic materials units. Confidence level of materials unit boundaries is indicated by either a solid (ground verified) or dashed (inferred) line.

Elements of the GRC Subsystem

Minimum elements of the GRC subsystem are:
- A display of geologic resources, usually located at an individual site, identified by symbol. Geologic resources include construction materials sources, ground water, caves, and geologic points of interest.

![Figure 4. Typical GRC map materials source information block.](image-url)
REILLY—APPLICATION OF GEOLOGIC INFORMATION IN FOREST MANAGEMENT

- A map display of geologic materials units (GMUs) distinguished by symbol. These units are designated by soil and/or rock type based on stratigraphy, physical characteristics, and associated processes. Certain soil types or landforms of a particular origin such as till or slope movement deposits may be designated as GMUs.
- A display of landforms, identified by symbol. Landforms displayed include rock outcrops and talus areas, wet areas, and slope features. The combination of geologic materials units and landforms is used to define geologic conditions.

Descriptive information concerning geologic resources and conditions is given in the map margin. It includes descriptions of the GMUs, an explanation of map symbols, a summary of information about construction materials sources, and a summary of special considerations such as significant geologic resources in the area.

Geologic Resources

The definition of geologic factors is found, in part, in planning regulations and the Forest Service Manual, Chapters 1910, 2550, and 2880 (U.S. Forest Service, 1985a). Energy and mineral resources were not added to this subsystem because they are already shown on the TRI Minerals subsystem maps.

The geologic resources shown on GRC maps reflect known occurrences. However, the fact that a resource is not displayed may reflect only a lack of past need or site-specific work to locate the resource.

Earth Construction Materials

This resource consists of deposits of rock, gravel, cinders, and mixtures of sand, silt, and clay that are of sufficient quality to be used as construction materials, such as fill, road base or surfacing rock, or armor for erosion protection. Since the overwhelming proportion of earth construction materials used on the GPNF is natural or crushed rock, the geologic resource is generalized as "rock resource".

Materials sources are inventoried and categorized as active or inactive. The status of the source is determined by the District Ranger after interdisciplinary analysis of the effects of source development. Management direction provided by the District Ranger consists of written criteria on size, period of operation, means of access, and ultimate rehabilitation of the site. Forest engineering geologists prepare a long-term source development and rehabilitation plan consistent with this direction. A contract operating plan is prepared, consistent with the development plan, designating that portion of the source to be used for each project.

By using the comprehensive process described above, impacts to GPNF resources are minimized, and each rock source is developed and used in an efficient manner.

Ground Water

This resource consists of known or inferred occurrence of subsurface water that can be developed for a potable water supply or other purposes. The groundwater resource in the GPNF has not been systematically assessed by geologists or hydrologists. The occurrence of groundwater and associated aquifer recharge systems is generally evaluated by GPNF geologists on an area or project basis. To date, this resource has been developed to supply recreation and administrative sites with potable water and for irrigation at the Wind River Nursery.

GRC maps show water well locations. GMU descriptions may also identify potential for groundwater extraction.

Caves

This resource consists of natural or developable underground spaces (caves). In the GPNF, numerous natural cave systems were formed by lava flows. The location of these caves is an important consideration for resource management activities because many contain significant geologic, cultural, or biological resources. Several caves provide recreational opportunities; however, some present hazards. Except for caves well known to the public that do not contain sensitive geologic or biological resources, cave entrances are not shown on GRC maps. Forest policy is to conduct an analysis to develop a management plan for any caves that might affect or be affected by management activities.

Geologic Points of Interest

This resource consists of examples of unique or spectacular features depicting geologic processes or phenomena of the Earth's evolution, including volcanic or glacial landforms and effects, academically significant contacts between soil and/or rock units, sedimentary structures, landforms, or fossil locations. Some geologic points of interest are regionally or nationally significant; all features inventoried may have value for scientific study, public education, and for recreation and interpretive programs.

As part of the forest planning process, all inventoried Geologic Points of Interest have been rated for significance using Forest Service criteria. The most significant features were considered for nomination as Special Interest Areas in the Forest Plan. The sensitivity to disturbance of sites was also rated to identify those features more likely to be damaged or destroyed by management activities. All sites will be analyzed to deal with the effects of adjacent management activities.

Geologic Conditions

This information category is defined to include features or processes that affect the safety, cost, or feasibility of resource management activities and development.
Geologic Materials Units

The GMU is a basic component of the GRC subsystem and is defined as an area characterized by a mappable distinctive geologic condition. Each GRC map area has been divided into one or more GMUs on the basis of soil and rock materials origin and characteristics (such as thickness, strength, and permeability), ground- and surface-water distribution, ground slope, dominant geologic processes (such as slope movement in the viscous fluid state), and characteristic landforms (such as benches or cliffs).

All materials unit descriptions follow the same format. To reduce subjectivity and increase reliability and reproducibility, standard soil and rock unit classification systems are used: the Unified Soil Classification System (USCS) (American Society of Testing and Materials, 1987), and the Unified Rock Classification System (URCS) (Williamson, 1984).

GMUs are differentiated on the GRC map by symbol and keyed to a description in the map margin. The description contains information about both the soil and rock materials, such as their thickness and physical properties. The description also contains a summary of the significant geologic condition for the GMU, such as the presence of the numerous slope movement features or high potential for slope movement (compared to adjacent GMUs), rock outcrop areas, and wet areas. In addition, the GMU description may contain information about the suitability of the materials for use in construction. In some instances, the description may recommend further site-specific work to evaluate areas in the GMU pertaining to potential management activities.

Information used to define GMUs is derived from a variety of sources of varied accuracy and reliability. The overall accuracy is suitable for general, preliminary management decisions. The two types of lines separating GMUs (solid and dashed) represent ground-verified and inferred boundaries, respectively.

Rock Outcrop

Rock outcrops affect the cost and feasibility of road construction and alternative timber harvest methods. This condition must be further analyzed because many of these areas have been inferred from air photo interpretation. Depending on physical characteristics, rock outcrop areas may exhibit different stability characteristics, may respond differently to management activities, and may have potential for use as construction materials.

Talus Deposits

Talus deposits are significant to timber harvest and transportation planning because of potential slope instability and difficulty in reforestation. Some talus deposits may be suitable for use as construction materials, depending on factors such as particle size, rock quality, ground slope, and deposit thickness. The value as a potential rock resource must be compared to other resource values, such as wildlife habitat.

Significant "Wet Areas"

This condition is defined as areas of elevated or seasonally high ground-water table or seep/spring areas. Since many of these wet areas are inferred from air photo interpretation, their location requires field confirmation, site-specific mapping of feature size, and determination of water quality. Wet area conditions affect road drainage and pavement design, the cost of road construction, and slope stability. Intensive field investigation prior to road design and construction is the basis for evaluating alternative design and construction techniques and mitigating slope stability concerns.

Slope Stability

Seven categories of slope movement forms and processes, distinguished by Roman numeral, appear on GRC maps. These seven categories represent three basic mechanisms (slide, flow, and fall) and four subcategories. Many features exhibit combinations of mechanisms. In those instances, the predominant mechanism is used as the basic descriptor.

The seven categories of mechanisms and associated symbols are a shorthand method of cataloging slope movement features. General guidelines for managing within GMUs cannot be applied to individual slope movement features. Management and resulting location and design alternatives must be based on site-specific field investigation and evaluation of conditions for each feature.

Subsystem Updating

As new information is acquired, geologists will make necessary changes to the appropriate GRC map. The revision date in the title block reflects new information, and a revised map is provided to the ranger district.

USE OF GEOLOGIC RESOURCES AND CONDITIONS MAPS

Potential Use of the GRC Subsystem by Resource Area Managers

Although the information displayed and described may relate most directly to engineering and slope stability concerns, GRC subsystem information is adaptable to many other national forest resource management needs.

The following examples are some potential applications of the GRC Subsystem information by resource area managers.

- Recreation management: Uses include interpretation for the public of geologic points of interest, development of caves suitable for public use, evaluation of potable ground-water potential, and consideration of effects of geologic processes on recreation developments.
• Fish and wildlife management: Slope failures contributing sediment to streams can be identified on GRC maps and stabilization alternatives developed.

• Range management: GRC maps may indicate potential for development of ground-water supplies for grazing allotments and wildlife improvement projects.

• Timber management: GRC maps, when used with other resource inventories, are useful in identifying nonproductive land areas or areas potentially not suitable for timber harvest. GRC maps are also useful for timber sale planning. By identifying areas of potentially adverse geologic conditions early in the planning process, time can be allowed for more thorough evaluation. Recommendations concerning potential mitigation can also be requested from Forest Service geoscientists. Slope stability may affect specific sale location, layout, and viability. Since, at this stage, timelines are established for various disciplines to provide further input to the planning process, the GRC map and summary can effectively convey significance of soil and rock factors at a preliminary resolution level.

During timber sale layout, the sale and transportation planners can use the GRC map to locate construction materials sources. The map is also useful for preliminary evaluation of geologic conditions prior to approval of spur road and landing locations. Identification of rock outcrop areas may suggest a viable opportunity to use rock bolts as logging system cable anchors (with appropriate verification).

GRC maps do not eliminate the need for further geospecialist involvement prior to project planning or design. The time involved in providing additional input is greatly reduced by concentrating on known problem areas recognized earlier in the planning process.

• Water and soil management: GRC maps are useful to soil and water specialists in evaluating site productivity, existing stability, soil parent material, and geologic factors relating to the production, transportation, and deposition of sediments affecting water quality. The GRC subsystem complements other resource subsystems such as soil and water, as conflicting information is identified and conflicts resolved.

• Minerals and geology management: GRC maps show the location of material sources. They may be used for preliminary evaluation of geologic conditions which may affect proposals to prospect, explore, or develop minerals or energy resources.

• Facilities engineering management: During initial transportation route planning, the GRC map provides a general estimate of how area slope stability may affect potential routes, as well as a basis for making a general estimate of cost of construction, by considering factors such as location of earth construction materials sources, location of potentially expensive rock outcrop or talus areas, location of recommended special construction areas, and materials properties such as ease of excavation. The GRC map is also useful in establishing preliminary estimates of feasibility of constructing associated transportation structures, such as bridges.

Examples of GRC Subsystem Use

Although the initial incentive for development of the GRC was support of forest planning, geologists tried to ensure that the information and format would be useful in the future. An example of the use of GRC information for three types of management needs is provided below.

Area Analysis

The Dry Burton Planning Area, in the northern part of the GPNF, is adjacent to private land in a scenic area. The steep slopes in the area have a history of natural slope movement, and the public expressed concern that any management activity in the area would increase the risk of a future catastrophic event. With the GRC map as a starting point, preliminary slope stability analyses were conducted using techniques developed to predict areas of high risk for slope movement from road construction and timber harvest. This allowed the District Ranger to be fully aware of the risk of management activities and allowed interdisciplinary analysis of the possibility of recommending certain portions of the area as unsuitable for timber harvest. The GRC map was also used to identify other geologic factors within the planning area. The resulting report (U.S. Forest Service, 1984) is an example of system application.

Environmental Assessment for a Proposed Road

Due to public responses during review of the draft Mount St. Helens National Volcanic Monument Comprehensive Management Plan (U.S. Forest Service, 1985b), the feasibility and environmental impacts of reconstructing Road 51 (a key route for access to the monument from the south and east) were assessed (U.S. Forest Service, 1986a). One of the management concerns identified in initial office evaluation for the project was the effect of road reconstruction. The area GRC maps were examined to estimate any adverse effects to geologic resources and conditions.

The following geologic information was obtained directly from the GRC maps for the project area.

**Geologic Resources**

• No inventoried geologic points of interest exist in the project area.
• Three caves marked on the area GRC maps are near the existing Road 51 and would not be affected by any proposed management alternative.
• All alternatives would use construction materials in varying amounts. Three materials sources are located on the GRC maps; one is within the project area.

Geologic Conditions

All route alternatives could potentially affect area slope stability. Alternative routes were evaluated for material suitability, natural and modified stability, and investigative complexity as estimated from the area GRC maps.

• Material suitability: The measure of suitability predicts the effect of the materials present on the timing, feasibility, and cost of construction of the alternative routes on the basis of the physical characteristics (permeability, foundation suitability and strength, excavation characteristics, and use of the material) of the earth materials present. The properties of the materials used were estimated from the GMU descriptions. Material suitability were rated as high, moderate, or low on the basis of these characteristics. Alternative routes traversing a majority of high-suitability earth materials were rated as of good suitability; routes traversing a majority of low-suitability materials were rated as of poor suitability.
• Natural stability: Some GMUs in the project area exhibit evidence of recent or past slope movement. Some portions of the area may be marginally stable due to materials characteristics and ground slope; other portions of the area are stable. Ratings ranged from good (no existing features; materials inherently stable) to poor (some slope movement features; inherently unstable material, especially on steep slopes).
• Modified stability: On the basis of material suitability, terrain, and alternative road prism designs, some areas have a higher probability of slope movement during or after road construction than do other areas. This measure predicts the effects of each alternative on the stability of excavations and embankments. Ratings range from good to poor on the basis of the estimated ability to eliminate slope stability problems along each alternative route. Areas in which there is a high probability of slope movement will require more stabilization measures in project design than those areas in which slope movements are unlikely.
• Investigative complexity: Field investigations may reveal conditions requiring special design or construction methods to insure safety and feasibility or to mitigate adverse impacts to other resources. An effect of each route alternative is the consequent variable degree of investigative complexity, which also carries a variable degree of uncertainty or risk to route safety, feasibility, cost, or performance. The complexity of the routes ranged from low (little or no steep terrain, no special structures or construction methods proposed) to high (steep, unstable or potentially unstable terrain, low-suitability earth materials, or one or more special structures proposed).

It is worth noting that this analysis began in the fall of 1985, and within a month the area began receiving snowfall. The environmental assessment was completed in early spring. Had the GRC maps for the planning area not been available, it would not have been possible to estimate the effects of alternatives on the geologic resources and conditions due to snow cover.

Project Design

Ideally, GRC maps are used in the initial planning stages of a project. If this is done, consideration of geologic factors would lead to a recognition of the potential impacts of a project on area geologic resources and conditions. After interdisciplinary analysis and management decision for the project alternative, an assessment can be made of the need for and degree of involvement of geoscientist personnel to participate in the design phase. Comparison of a proposed project location with the area GRC map will reveal areas where more intensive site-specific investigation is warranted. After project completion, any additional information on area geologic resources and conditions gained during project area investigations is added to the GRC map to improve its overall accuracy.

SUMMARY AND CONCLUSIONS

Prior to 1980, the GPNF, like most national forests, did not have a means to store and display geologic information. Consequently, involvement of the geoscientist did not occur until the project design stage, when changes or mitigation of adverse impacts were difficult or prohibitively expensive and time consuming. If a geologic information system that was easy to use, readily updated, and technically valid could be developed, use of geologic information would increase. With greater information availability, involvement would shift from project design to forest-wide planning and area analysis. If such a shift could occur, it was predicted that cost of project investigation and design would be reduced, and unforeseen environmental consequences would also diminish.

In late 1979, the opportunity arose to develop such a system with the appointment of the Forest Geologist to the Forest Planning Interdisciplinary Team. A proposal was accepted to develop a long-term geologic informa-
The GRC system was built on accurate topographic maps, making spatial location of features and digitizing of the location of materials boundaries and geologic resources and conditions a possibility in the future. This makes the GRC system fully compatible with Geographic Information System concepts and software.

The GRC map system is easy to construct. Once agreement was reached on format and materials classification systems (fully documented in a User Guide), the system was implemented by starting with project file information, published geologic reports, and air photo interpretation. As opportunities arise, GRC maps can be field verified and supplemented with additional information. With time, the maps will increase in accuracy.

REFERENCES


In some areas, such as the Dog Hair Timber Sale Area, Quilcene Ranger District, Olympic National Forest, it is not economical to build new roads and harvest timber because the source of rock road material is far from the site and the mature timber is not large enough for the saw mill. One solution to this problem is the use of wood chip as road base course material. By processing wood chips onsite, this material can be transported ready for a pulp mill and the remainder can be used in road construction. In this U.S. Forest Service photograph, the bulldozer is moving woody debris before spreading more wood chips. A small volume of crushed rock aggregate can be used to cap the woody base course. Photograph courtesy of the U.S. Forest Service.

One means of stabilizing a logging road that has failed is the earth-reinforced structure. In this Forest Service photograph, a load of rock aggregate is about to be unloaded on top of steel "ribbons", visible behind the truck. The ribbons are "nailed" in place and serve as anchors for the wall. After several lifts are placed, the road will be reestablished at its former elevation. (uphill of the truck).
SECTION 3

SITE INVESTIGATION

Principal contributors:

Mark Leverton, Engineering Geologist
(Section Leader)
USDA Forest Service
Willamette National Forest
South Zone Engineering
49098 Salmon Creek Road
Oakridge, OR 97463

Ken Baldwin, Engineering Geologist
USDA Forest Service
Klamath National Forest
Happy Camp, CA 96039

Tom Koler, Engineering Geologist
USDA Forest Service
Intermountain Research Station
1221 S. Main
Moscow, ID 83843

Dennis Larson, Assistant Forest Engineer
USDA Forest Service
Olympic National Forest
1835 Black Lake Boulevard SW
Olympia, WA 98507-5623

Mike Long, Forest Engineering Geologist
USDA Forest Service
Willamette National Forest
P.O. Box 10607
Eugene, OR 97440

Ken Neal, Engineering Geologist
Kenneth Neal & Associates
2014 Baker Terrace
Olympia, WA 98501

Rod Prellwitz, Geotechnical Engineer
USDA Forest Service
Intermountain Research Station
1221 S. Main
Moscow, ID 83843

Richard VanDyke, Geotechnical Engineer
USDA Forest Service
Siskiyou National Forest
Westside Engineering Zone
93976 Ocean Way
Gold Beach, OR 97444

Doug Williamson, Forest Engineering Geologist (Retired)
USDA Forest Service
Eugene, OR 97402
Section 3. Site Investigation

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3A. Problem Definition and Application of the Scientific Method

Thomas E. Koler, Research Engineering Geologist, Intermountain Research Station
Kenneth Baldwin, Engineering Geologist, Klamath National Forest

3A.1 Introduction

This section describes how a geotechnical specialist can organize a project, using the scientific method, to study a slope stability problem. The following parts of this section contain the actual tools and methods (i.e., classifications, surface and subsurface investigations, and monitoring). The process consists of the specialist defining the problem of interest (what is requested to be done), constructing a methodology to solve the problem (how to get the requested product), the serial order of steps in the methodology (timeframe and people involved), and costs to get the project done. Intrinsic to this is the need to calculate whether it is worth the time and money to carry out the work (e.g., an estimated monetary value analysis as described in section 6). Sometimes land managers do not understand the effort involved in getting a product completed for them. If they know the costs in advance, it will save you the problems inherent in unfulfilled low-cost expectations.

In graduate school this project setup is called a thesis proposal, but, because we are not part of academe, we use other names for it. In the Pacific Northwest Region, and in the private sector, many of the geotechnical specialists call it a job prescription. Regardless of what it is called, it is the blueprint for project work, usually including purpose and scope, methodology and analysis, scheduling, report format, and funding. We will describe these parts individually. Figure 3A.1 illustrates the three-level stability analysis process, with the portion covered in section 3, the level III data base, highlighted.
3A.2 Purpose and Scope

Within the scientific method framework, the purpose and scope section of the job prescription is the postulate you will be examining. In this section you will describe the problem to be examined and what is being requested, the location and areal size of the project, and the scale of the field data collection. Because you are interpreting the customer's request, this is the most critical part of the job prescription; therefore, communication has to be successful. Frequently, what the customer needs is unclear, and you will need to ask for a clarification. For example, a timber planner will ask for a slope stability analysis within a particular portion of the proposed timber sale area. Is this to be a site-specific investigation or a basin-wide stochastic investigation? In this example it is probably a basin-wide stochastic investigation, although the planner asked for a stability analysis in only a few hundred acres of the planning block. If that is the case, then a level I stability analysis is the appropriate tool, but with inadequate communication you could have completed a level III analysis instead. Make sure you understand what the postulate is and what the customer needs. This is the time to reach agreement on what the customer needs, and how you will meet those needs.

3A.3 Methodology and Analysis

The next logical step in the scientific method—and for the project prescription—is to test your postulate. How are you going to approach the project and what type of data do you think you will be gathering? Are the data collected at one scale or several scales? What types of parameters will you be collecting data for and how will you analyze them? To complete the project successfully, you must lay out the system and method to be used. You can still change your methodology as you progress with the project, but you need to know your starting "pathway," otherwise you will collect a plethora of data with no rhyme nor reason. Data should be analyzed preliminarily as they are received in order to check that they answer the
questions for which they are being collected. For example, at the outset of drilling, we develop a plan for the location of the drill holes. Many times after a few holes have been drilled, new questions arise. If the top of the bedrock dips steeply across the width of a landslide, then it may be beneficial to concentrate drilling on the side with the deeper soil. Finding a significant aquifer in an early drill hole may expose the need to better define that aquifer, rather than to continue drilling according to the original plan.

3A.4 Scheduling

In the scheduling section, describe who is going to do what and during what time-frame. Are you doing reconnaissance work that will take 2 days, or are you doing site-specific work that will take a crew of four people 2 weeks to complete?

3A.5 Report Format

What type of document does the customer need? Sometimes all that is needed is a two- or three-page trip report. At other times, the customer may ask for a full geotechnical report. All reports should have purpose and scope, methodology and analysis, discussion of findings, conclusions and recommendations, and references sections, with appendixes included as appropriate. Remember, the purpose of the paper trail is to allow you or others to go back later and see what was requested and what was done. The report should be thorough, professional, and have a letterhead or cover sheet. A superior report is well worth the effort.

3A.6 Funding

Your customer will be very interested in this section of the job prescription, especially if you are using his or her funds. You probably will not be able to estimate the exact dollar amount because of changes in how the work gets done, but you should be able to get within 5 to 10 percent of the actual costs.

Who should get a copy of the job prescription? Your supervisor and the customer are clearly due a copy. You should also keep a copy in a field file and an office project file. The field crew may also need copies if you are not going to be on the site every day. Before you distribute copies, however, you may want to review a draft copy with the customer to clarify details and make any required changes.

NOTE: See appendix 3.1 for a complete discussion of the scientific method.
3B. Soil and Rock Classification for Engineering Analysis and Design

Thomas E. Koler, Research Engineering Geologist, Intermountain Research Station

3B.1 Soil Classification

Several soil classification systems are available to the geotechnical specialist—including the Unified Soil Classification System (USCS), the U.S. Department of Agriculture (USDA) system, and the American Association of State Highway and Transportation Officials (AASHTO) system, each of which will be described in this section. All three classifications use grain size, but the USCS and AASHTO systems also include physical properties of the soil, whereas the USDA system does not. Therefore, most geotechnical specialists use the USCS or AASHTO system in their work.

3B.1.1 Unified Soil Classification System (USCS)

The USCS is the most popular soil classification system among geotechnical engineers (Dunn et al., 1980). Casagrande (1948) developed this system as the Airfield Classification System. In 1952, it was adopted with minor modifications by the U.S. Army Corps of Engineers and the Bureau of Reclamation. The American Society for Testing and Materials (ASTM) adopted this system as a standard method (ASTM D-2487) for classification of soils for engineering purposes. Howard (1986a and 1986b) modified the USCS for the Bureau of Reclamation, and this modification was adopted by ASTM in 1987. The visual and laboratory versions of the USCS described in appendix 3.2 of this guide are excerpted from Howard's Bureau of Reclamation work.

Two figures that are not included in Howard's modification of the USCS are shown below. Figure 3B.1 is a comparison of grain sizes between the USCS, AASHTO, and USDA systems. You may find this useful in your communication with soil scientists and hydrologists not familiar with the USCS. Figure 3B.2 is an information sheet for identifying soils in the field. A photocopy of this inside a clipboard is frequently useful for the field geologist and engineer.

3B.2 Rock Classification

In the geological sciences there are several different types of rock classifications. In the geotechnical world, the rock classification systems have one thing in common: they provide information gathered by engineering geologists in a format usable by engineers for design applications. What purpose does it serve if an engineering geologist classifies a rock outcrop only as a tholeitic basalt or an orbicular granodiorite? Will this help the geotechnical engineer with his or her design? Clearly the answers to these questions are "none" and "no." It is our responsibility as engineering geologists to communicate with the engineers in a descriptive manner that will help them in developing a cost-effective and safe design.
### An effective rock classification will include rock type name (simple names, such as sandstone, basalt, granite, and schist are adequate); texture; weathering (including color for fresh and weathered surfaces); stratigraphic position; discontinuity description (orientation, surface roughness, openings, spacings, and fluid transmittal); unconfined compressive strength (relative hardness); and unit weight.

Other pieces of information that could be included, but are not absolutely necessary, are mineralogy, fossils, and formation name.

The description of rock classification located in appendix 3.3 is from the Oregon Department of Transportation's soil and rock classification manual.

#### 3B.2.1 Unified Rock Classification System (URCS)

There are several geotechnical rock classification systems available to the engineering geologist. Most require rock core samples and laboratory tests that can be quite expensive. The classification system described in appendix 3.4 is field-based and has operational definitions which do not require any equipment more sophisticated than a cloth measuring tape, a Brunton compass, a ball-peen hammer, and a small weighing scale.
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<th>Group Symbols</th>
<th>Typical Names</th>
<th>Information Required for Describing Soils</th>
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<tr>
<td>Predominantly one size or a range of sizes with some intermediate sizes missing</td>
<td>GP</td>
<td>Poorly graded gravelly-sand-silt mixtures</td>
<td>Give typical name; indicate degree of character of plasticity and sand and gravel, maximum size, shape, and roundness of gravel in the coarsest fraction, and hardness of the coarsest fraction; name and other pertinent descriptive information and symbols in parentheses.</td>
</tr>
<tr>
<td>Plastic fines (for identification procedures, see CL below)</td>
<td>GM</td>
<td>Silty sands, poorly graded gravel-sand-silt mixtures</td>
<td>For undisturbed soils, add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.</td>
</tr>
<tr>
<td>For undisturbed soils, add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.</td>
<td>GC</td>
<td>Clayey gravelly-sandy clayey mixtures</td>
<td>Example: Silty sand, gravelly about 20%, hard, angular gravel particles 1/8 in. in size; rounded and nearly spherical large gravelly sands, little or no fines</td>
</tr>
<tr>
<td>Wide range in grain sizes and substantial amounts of all intermediate particle sizes</td>
<td>SW</td>
<td>Well graded sands, gravelly sands, little or no fines</td>
<td>For undisturbed soils, add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.</td>
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<tr>
<td>Predominantly one size or a range of sizes with some intermediate sizes missing</td>
<td>SP</td>
<td>Poorly graded gravelly-sand-silt mixtures</td>
<td>For undisturbed soils, add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.</td>
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<td>Example: Clayey gravelly-sandy clayey mixtures</td>
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<td>Clayey sands, poorly graded sand-clayey mixtures</td>
<td>Example: Clayey gravelly-sandy clayey mixtures</td>
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### Identification Procedures on Fraction Smaller than No. 40 Sieve Size

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<td>Quick to slow</td>
<td>None</td>
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<td>Medium to high</td>
<td>None to very slow</td>
<td>Medium</td>
</tr>
<tr>
<td>Slight to medium</td>
<td>Slight</td>
<td>Slight</td>
</tr>
<tr>
<td>Slight to medium</td>
<td>Slow to none</td>
<td>Slight to medium</td>
</tr>
<tr>
<td>High to very high</td>
<td>None</td>
<td>High</td>
</tr>
<tr>
<td>Medium to high</td>
<td>None to very slow</td>
<td>Slight to medium</td>
</tr>
<tr>
<td>Medium to high</td>
<td>Ready to identify by colour, odour, spongy feel and frequently by fibrous texture</td>
<td>Ready to identify by colour, odour, spongy feel and frequently by fibrous texture</td>
</tr>
</tbody>
</table>

### Laboratory Classification Criteria

\[
C_U = \frac{D_{60}}{D_{10}}
\]

Greater than 4

\[
C_C = \frac{(D_{60})^2}{D_{10} \times D_{60}}
\]

Between 1 and 3

Not meeting all gradation requirements for GW

Afterberg limits below "A" line, or PI less than 4

Above "A" line with PI between 4 and 7 |

Afterberg limits above "A" line, with PI greater than 7

Above "A" line with PI between 4 and 7 and borderline cases requiring use of dual symbols

### Field Identification Procedures

- **Typical Names**: WELL-GRADED GRAVELS, GRAVEL-SAND-MIXTURES, LITTLE OR NO FINES
- **Information Required for Describing Soils**: Give typical name; indicate approximate percentages of sand and gravel, maximum size, shape, and roundness of gravel in the coarsest fraction, and hardness of the coarsest fraction; name and other pertinent descriptive information and symbols in parentheses.

### Field Identification Procedures

- **Typical Names**: POORLY GRADED GRAVELY-SAND-SILT MIXTURES
- **Information Required for Describing Soils**: For undisturbed soils, add information on stratification, degree of compactness, cementation, moisture conditions, and drainage characteristics.

### Field Identification Procedures

- **Typical Names**: SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES
- **Information Required for Describing Soils**: Example: Silty sand, gravelly about 20%, hard, angular gravel particles 1/8 in. in size; rounded and nearly spherical.

### Field Identification Procedures

- **Typical Names**: CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

### Field Identification Procedures

- **Typical Names**: GRAVELY SANDY CLAY, SEDIMENTARY CLAY
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

### Field Identification Procedures

- **Typical Names**: CLAYEY SANDY CLAY, CLAYEY SANDY Silt
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

### Field Identification Procedures

- **Typical Names**: CLAYEY SANDY CLAY, LOESS
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

### Field Identification Procedures

- **Typical Names**: CLAYEY SANDY CLAY, CLAYEY SANDY CLAY Silt
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

### Field Identification Procedures

- **Typical Names**: CLAYEY SANDY CLAY, CLAYEY SANDY Silt
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

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- **Typical Names**: CLAYEY SANDY CLAY, LOESS
- **Information Required for Describing Soils**: Example: Clayey gravelly-sandy clayey mixtures.

### Field Identification Procedures

- **Typical Names**: CLAYEY SANDY CLAY, CLAYEY SANDY CLAY Silt
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3C. Surface Investigations

3C.1 Synopsis of Geotechnical Methods

Thomas Koler, Research Engineering Geologist, Intermountain Research Station

3C.1.1 Introduction

Sections 3C.1 and 3C.2 are a compilation of several unpublished notes and handouts from Douglas Williamson and Dennis Larson. Section 3C.1 is a description of what Williamson (1979) has listed as the basic elements for a slope stability assessment using a surface investigation. In section 3C.2, definitions and a review of fluid mechanics have been added to Larson’s checklist (Larson, 1978) to describe the effects of surface and subsurface water flow on slope stability conditions. Appendix 3.5 is a paper by Williamson, Larson, and Kenneth Neal, presented at the Association of Engineering Geologists annual meeting in 1991, describing how an engineering geologist or a geotechnical engineer, skilled in detailed geologic interpretation, can portray subsurface conditions from surface measurements.

3C.1.2 Basic Elements of Slope Stability Assessment

Williamson (1979) developed a checklist for the geotechnical specialist to use when working on a geotechnical project. I have taken his checklist and added my commentary to each item. Williamson’s checklist is in sequential order so that the specialist will know which steps to follow from start to finish of the project.

1. Purpose of assessment.

A geotechnical specialist needs to identify the application of the assessment before proceeding with a surface investigation. Field work should be completed at a level appropriate for the intended use. Requests are made to answer a question or provide data for a specified application. The geotechnical specialist must prescribe a scope of services commensurate with the need. Sometimes the specialist needs to ask the requester for additional information about what is wanted. For example, the requester may be asking for a reconnaissance-level project when it would be better to provide a site-specific project report. The geotechnical specialist should understand what the customer wants before the investigation is started. If there is a misunderstanding, the specialist will certainly find out during the customer feedback.

2. Sequence of events (slope history).

The axiom “the key to the geologic past is the present,” developed by James Hutton in the 18th century, is useful in assessing slope stability. Understanding what has occurred on the slope over geologic time helps in predicting what will happen in the future. In addition to examining the geologic history of a slope, we need to look at the history of development (e.g., engineering and logging). Did slope instability occur following development? Can cause/effect relationships be identified? A quick
check of previous geotechnical/geologic reports and maps and a careful review of historic aerial photographs helps in assembling a slope stability history. It also is helpful to ask the local road maintenance crew what they know about the slope in question. The individuals who keep the road access open usually know a great deal about slope stability in an area. In addition, colleagues working in engineering zone offices, ranger district offices, and forest headquarters usually have information about the slope stability history. A few telephone calls to hydrologists, soil scientists, foresters, and other resource specialists working in the forest usually generate additional information about the slope stability history.

3. Topography, elevation, slope, relief, and landform.

Together, the topography, elevation, slope, relief, and landform are descriptive in assessing a slope for stability. When evaluating one, the geotechnical specialist needs to look at the others in this assessment. Also, these elements can be used for slope stability information that the requester can use for his or her work (e.g., proposed timber sale, proposed road construction, and road reconstruction or closure).

Topography

A topographic map helps identify areas with potential instability. For example, areas that are concave usually concentrate more subsurface water than do planar or convex slopes (Dunne and Black, 1970). These concave slopes may indicate previous slope movement where the head of the slide has moved downslope, leaving a bowl-shaped escarpment (Benda and Cundy, 1990). Also, slope areas that are hummocky may indicate a complicated relationship between ground water gradient, transitory ground water movement, and soil units (Iverson and Major, 1987). Topographic maps may not show areas of hummocky slopes for a level I stability analysis because of the map scale (e.g., 1:24,000), but the hummocky topography should be discernable at level II and III stability analysis map scales (e.g., 1:3600 to 1:120).

Elevation

Similar elevations for landslide features—such as headwalls, sag ponds, springs and seeps—are common for areas that have more than one unstable slope. A topographic map with a landslide overlay usually provides enough information during a level I stability analysis to identify areas with a common elevation for a particular landslide feature. By building an engineering soil overlay during the level I stability analysis, the specialist usually can locate similar materials at common elevations. Information in this soil overlay may come from previous geotechnical or geologic reports, geologic or soil survey maps, colleagues, road maintenance crews, or reconnaissance field trips. For example, river valleys that were recently covered by glaciers may have a stratigraphic order of glacial deposits from the valley floor up the valley walls. This stratigraphic sequence may look like the information given in table 3C.1; however, the stratigraphy is rarely horizontal as given in this example. Elevation information gathered and evaluated during the level I analysis can be used for levels II and III. Using the level I analysis elevation data, the geotechnical specialist can help the transportation planner to locate proposed road corridors (e.g., finding a corridor that will miss potential slope stability problem areas or that will cross the slope on a landslide providing the smallest risk). For design work in level III stability analysis, these data may provide useful information. For example, the
spring at an elevation is probably associated with a slope feature a certain number of feet upslope or downslope and may be a key factor in the level III analysis.

Table 3C.1.—An example of common soil units and slope stability features for elevation ranges used in levels I, II, and III stability analyses.

<table>
<thead>
<tr>
<th>Elevation (Feet)</th>
<th>Unified Soil Classification</th>
<th>Slope Stability Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>2,100–2,400</td>
<td>Soil Unit A: MH, SILT; gray, wet, above the plastic limit, medium toughness, slow dilatancy, medium dry strength, stiff consistency. Origin: Glacial lake deposit.</td>
<td>Slump-earthflow headwalls and sag ponds.</td>
</tr>
<tr>
<td>1,850–2,100</td>
<td>Soil Unit B: SM, SAND with silt; brown, moist, nonplastic, dense compaction. Origin: Glacial outwash.</td>
<td>Slump blocks.</td>
</tr>
<tr>
<td>1,500–1,850</td>
<td>Soil Unit C: GP, GRAVEL with sand; brown, wet, nonplastic, dense compaction. Origin: Glacial fluvial, glacial outwash.</td>
<td>Slump-earthflow toes. Debris flow headwalls, springs, and seeps.</td>
</tr>
<tr>
<td>1,300–1,500</td>
<td>Soil Unit D: GM, GRAVEL with silt and sand; wet, brown, nonplastic, loose compaction. Origin: Slope deposits and debris flow fans.</td>
<td>Debris flow channels and fans.</td>
</tr>
</tbody>
</table>

One useful method is to mark on a topographic map areas that form bench-shaped areas. Williamson (1987) found a cyclical relationship between the elevations of topographic flat areas and valley development in western Oregon. He found that bench-shaped features can be found with great regularity at the following elevation intervals (in feet): 300–350, 600–650, 1,000–1,200, 1,700–2,000, and 2,800–3,200. Williamson associated the 300–350 and 600–650 foot intervals with effects from the Missoula flood. In his field work he has found that the 1,700–2,000 foot elevation interval goes across the Oregon Coast Range and probably the Willamette River valley to the west flank of the Cascade Mountain Range where it is commonly found on Forest Service lands. The bench-shaped features found at these intervals were developed when the valley floor was at a higher elevation than today. These features are not necessarily river terraces, but commonly a combination of both river and slope processes (e.g., a debris flow fan overlying a river deposit). Williamson is not the only one to identify these topographic features.

Similar base-level elevations can be found in most mountainous areas. Diller (1902) found several flat surfaces produced by erosion in the Klamath Mountains and described them as peneplains. Williamson described these as river valley stage development. Allen (1986) provided an excellent summary of Diller's observations and gave additional information from his own field work that also supports Williamson's work. All three geologists found the same elevation intervals, although Williamson found some higher than the others did.
By building a map overlay for the topographic flat areas in a study area (e.g., a level I stability analysis for a proposed timber sale analysis area) and superimposing the overlay onto the engineering soil unit and landslide overlays, the specialist usually will find an association between topography, engineering soil units, and areas of slope movement. This information can then be used in the LISA (Level I Stability Analysis 1991) computer program to evaluate the probability of slope movement in the study area. This same information can also be used in the level II stability analysis when the geotechnical specialist is working with the transportation planner evaluating a proposed road corridor.

Slope

Changes in ground surface slope angle reflect differences in physical characteristics of soil and rock materials or the presence of water. Slope changes observed at the same location over time, when reviewing aerial photographs, usually indicate either slope movement or significant erosion. These changes are most readily seen in aerial photographs taken over several years or decades. Slope angle for unconsolidated and noncohesive materials usually will give a close approximation for $\phi$, the internal angle of friction. If the engineering soil units are known or even estimated for the study area, the geotechnical specialist may be able to assign a $\phi$ value without completing a field or laboratory strength test by measuring the slope angle from the topographic map or aerial photograph. However, these values are sufficiently accurate only for a level I stability analysis and certainly are not appropriate for consolidated and cohesive engineering soils. Therefore, great caution should be used in associating slope angle with $\phi$ at this level of analysis. For levels II and III stability analyses, the specialist should collect field samples for determining $\phi$ and not depend upon the possible slope angle--$\phi$ relationship.

Relief

Relief is the overall change in elevation within an area. Relief helps tell the slope stability story for a study area for levels I and II stability analyses. Is the topography flat, so that there is very little relief, or is it steep and therefore has high relief? What are the geologic processes that are resulting in the relief in the study area? Also, what is the aspect (slope direction) for various types of relief (low, medium, high)? Is this association between aspect and relief type controlled by bedrock structure (e.g., folding and faulting), or is it a geomorphic landform (e.g., landslide), or a combination of both? Are there certain types of slope movements associated with a relief type and underlying bedrock structure (e.g., rock topple in high relief slopes composed of vertically jointed granite)? Are there changes in relief within the study area? What is occurring at those changes? Are those relief changes in areas where the slope has moved?

Landform

Geologic processes use soil, rock, and water to create landforms (e.g., glacial, volcanic, fluvial, eolian, and lacustrine). Landforms can be described by geologic process, elevation, topography, slope, and relief. How do these control the development of the landform? Has this particular type of landform evolved over time from one type to another? What is the geologic/geotechnical story for a landform with unstable slopes?
4. Weather, climate, rainfall, and temperature.

Changes in meteorological conditions—weather, climate, rainfall, and temperature—will affect slope stability conditions (see section 2). Unfortunately, records in the Western states for these conditions go back only 100 to 150 years, so it is impossible to determine whether the current conditions are unusual or normal for a long period of time. However, we can use data collected over decades to make some comparisons for short time periods (decades or half centuries). These data are probably the most useful in the level I stability analysis, specifically for input to the LISA computer program ground water file. It is also possible to use rainfall information in back-calculating neutral effective stresses in the levels II and III stability analyses.

Slope stability conditions can also be influenced by changes in the vegetation canopy that intercepts rain and snow. Prior to timber removal by harvesting or fire, precipitation reaches the ground by throughfall (precipitation moving through the canopy) and stemfall (intercepted precipitation that moves down the tree stem to the trunk and down to the ground). Interception, throughfall, and stemfall cease when timber is removed from the slope. The rain-on-snow phenomenon (Harr, 1981) is a result of this canopy removal in areas where there is a transient snow zone. Snow that is normally intercepted in the tree canopy is now on the ground and has the potential of melting rapidly if rained upon. This phenomenon continues until the canopy is re-established, usually 30 years after timber harvest. When rainfall occurs on a snowpack overlying an unstable slope, there is a potential for a significant increase in slope instability because of the potential increase in ground water. This will occur when there is a sudden influx of water into the soil as the snowmelt and the rain infiltrate into the soil column. Therefore, an evaluation for the potential of rain-on-snow events on an unstable slope should be included in a slope stability assessment.

5. Vegetation.

The type and location of vegetation should also be included in all three levels of slope stability assessment. Areas where there are hydrophilic species should be recorded and included in each assessment. Also, asking an ecologist or forester about the fire history in the study area is important since vegetation types are affected by fire and contribute to the slope instability by the loss of root shear strength.

In level I stability analysis, the geotechnical specialist usually uses aerial photographs in observing changes in the vegetation canopy. Typical questions the specialist asks at this stability analysis level are: What is the pattern in the vegetation change? Is this pattern similar to one for various types of slope movements and the time of those movements? Are there hydrophilic species located on the photographs that would indicate areas of saturation? How does this information tie in with the topographic and elevation information?

In levels II and III stability analyses, the specialist needs to evaluate the role of vegetation in slope stability at a more site-specific scale than in level I. For example, the pattern of the vegetation on and adjacent to the slide mass may be important in understanding better if, how, and why the slope has moved. Questions regarding direct relationships between vegetation and slope movement can also be answered. For example, careful site observation will provide information on whether tree roots
are providing some anchoring of the slide mass to underlying rock or whether vegetation is forming a network across the top of the slide mass to more stable ground (much like a large net holding the mass in place). Indirect relationships, such as subsurface drainage characteristics, may also be indicated by vegetative pattern. A geotechnical specialist may have a good handle on the physical strength properties of the slide mass, but this information may be moot if the specialist has not evaluated the role of the vegetation for the site in question.


Understanding the distribution of surface water is important at all three levels because the movement of water is a critical part of whether a slope will be stable. The easiest way to get a quick interpretation for surface water distribution is to look at drainage patterns on a topographic map. These drainage patterns reflect underlying bedrock structural characteristics within the project area for timber sale and transportation planning. Typical drainage patterns the geotechnical specialist may be looking for on a topographic map in a level I or II stability analysis include dendritic, parallel, trellis, rectangular, radial, annular, multi-basinal, and contorted (Howard, 1967).

**Dendritic**

The dendritic pattern is tree-like in form. Usually a dendritic drainage pattern indicates that the underlying bedrock is uniform, and there is no dominant control over the flow direction. Commonly this pattern is found in sedimentary or igneous bedrock that has a nearly horizontal dip.

**Parallel**

The parallel pattern is commonly found on bedrock that is tilted steeply in one direction so that the streams are forced to flow in one direction.

**Trellis**

The trellis pattern resembles a grape trellis with small branches attached to elongated branches. This drainage pattern is common in folded bedrock where the elongated stream courses are forced to travel along valleys underlain by nonresistant material, usually along the bedrock strike. The tributaries to these relatively longer streams are at right angles to the main stem and usually flow down bedrock dip.

**Rectangular**

Discontinuities (e.g., fractures, joints, and faults) in bedrock can control the drainage pattern in a study area. When this occurs, the drainage pattern can be rectangular because the discontinuity geometry forces tributaries to flow into main-stem streams at or near right angles. This pattern is different from the trellis pattern because the tributaries are usually not as short as those in the trellis pattern.

**Radial**

When streams flow from a common center, usually a mountain, the resulting drainage pattern is radial. This drainage pattern is common in volcanic mountain ranges.
Annular

Streams form concentric rings around a common center in the annular drainage system. This is common in the Southern states where there are salt diapers that form structural domes around which this drainage system is formed.

Multi-basinal

In karst areas where large sink holes are common, the multi-basinal drainage pattern is dominant. Because the carbonate rock in karst commonly goes to solution when exposed to ground water, the resulting drainages can be isolated from one another. The collection of these individual drainages is referred to as multi-basinal.

Contorted

In complicated metamorphic terrane the underlying structure is complex; therefore, the drainage pattern is commonly contorted. This type of drainage pattern is common in mountains where the metamorphic core is exposed.

7. Surface distribution of soil and rock units.

Within the three-level system, the geotechnical specialist should spend time driving or walking across the project area before any mapping occurs. Through this reconnaissance, the specialist gets a feel for the surface distribution of the soil and rock units. The amount of time spent doing this overview of the project area depends upon the area's size, geological complexity, and logistical characteristics. The time spent reconnoitering in a level I project area is typically 2 or 3 days if the project is 5 to 10 square miles in size. For a level II project area, this quick assessment will take about 1 day for a 1- to 2-mile-long proposed road corridor. In a level III site, the reconnaissance is usually several hours.

Once the geotechnical specialist has completed an overview of the area and knows what the soil and rock units are (at this point they should be classified), then the mapping of the surficial distribution can begin. The level of detail portrayed in mapping soil and rock units is scale-dependent. For a level I project the contact boundaries between soil and rock units may not be very accurate, but for a level III project these margins will be accurate to within a few inches. Level II project mapping has a level of detail similar to level III, but with less scalar accuracy.

8. Subsurface distribution of soil and rock units.

The goal in obtaining the surface distribution of soil and rock units is to be able to use these data to predict the location and geometry of the units under the surface and the subsurface physical relationships to form and process. This can be completed by using the strike and dip of the rock (and sometimes the soil) unit, the depositional or erosional form of the material (e.g., lenticular, lobate, tabular, etc.), estimated moisture content, unit elevation at one location relative to the same unit at another location, and stratigraphic measurement of these units. In appendix 3.5, this method of measuring and projecting soil and rock units is explained in detail.
9. Representative sampling of surface and subsurface soil and rock units.

Collecting samples that are representative of the soil and rock units in a slope stability assessment is important to give the specialist a reference sample for comparison of one unit material to another. Also, having a representative sample is useful if there are several people involved in the project identifying and measuring the units. Mistakes can happen when classifying material, so having a sample to refer to is helpful. Also, during the analysis stage of a project, it is always good to have typical examples available for a quick reference.

What are representative samples for levels I, II, and III? Rarely in a level I stability analysis will the geotechnical specialist have all the typical examples for the project area, especially if the project timeframe is short and the area large. At this point in the project, the specialist should know where there are gaps in the knowledge and the areas that are probably most unstable or have the potential to become unstable. If this is the case, then the specialist can concentrate field efforts in those areas to fill in the information gaps. This approach can also be used for a level II project. At this level of work, the specialist usually divides a proposed road corridor into segments that have similar materials and conditions. This way the specialist can determine which segments probably need the most attention; hence, most of the representative samples are collected in these areas. Project work in a level III stability analysis is site-specific; therefore, all soil and rock units can be representatively sampled.

10. Subsurface exploration.

Normally, subsurface exploration is done in level III when the geotechnical specialist needs to confirm his or her subsurface interpretation or when subsurface testing or sampling requires accurate strength values for analyses. Usually this confirmation is needed for projects that will have an expensive structure constructed or when there is concern for public safety. Analysis of expected monetary value is normally made to determine the best method of subsurface exploration. Methods of subsurface exploration include mechanical methods (drilling, augering, and backhoe trenching), hand explorations (hand auger, soil probe, or drive probe), and geophysical surveys (seismic, magnetometer, etc.). Geophysical methods are used for all three levels but are usually not as accurate as the mechanical or hand methods.

11. Subsurface water distribution and pressure.

Subsurface water distribution and pressure are analyzed in all three levels of stability analysis but more commonly in levels II and III than level I because pressure head is more easily identified and evaluated at these levels. The geotechnical specialist does not always need subsurface exploration data to determine the water distribution and pressure. Usually surface data can be identified and measured to help portray subsurface water conditions (as described in appendix 3.5). Some of this interpretative work is fairly obvious; for example, a concave slope covered by coarse-grained soils and underlain by bedrock with no discontinuities has the potential to have subsurface water. The difficult part in analyzing the condition of the ground water is the time (steady or unsteady state) and depth (uniform or nonuniform) variables tied to rainfall duration-intensity and the infiltration of this surface water through the coarse-grained material. This rainfall/ground water relationship can be estimated using a method developed by Sangrey et al. (1984) if the geotechnical
specialist has precipitation and water well data. However, much work needs to be
done to understand time and depth variables for soils that are periodically unsaturated
(Koler, in prep.). Until this work is completed, the specialist should use all of the
available data to build a subsurface model (e.g., flow net or finite difference) to
evaluate the ground water distribution and pressure. Section 4E provides the
instructions for building a flow net model.

12. Testing for classification and strength of soil and rock units.

Representative soil and rock unit samples are classified in the field. The most
common classification systems are the Unified Soil and Rock Classification Systems
described in sections 3B.1 and 3B.2. The geotechnical specialist may want to collect
field samples for laboratory testing if the soil or rock unit has a low strength value
identified in the field classification process.

For a level I stability analysis, the soil and rock unit classifications are estimated
from existing information (e.g., maps, aerial photographs, project files, and reports)
and field reconnaissance. Sometimes field samples are collected at this level for
additional testing in the laboratory, usually for obtaining angles of internal friction
and cohesion. In these cases, the geotechnical specialist can collect the sample using
a Shelby tube (if the soil is not too coarse-grained). For levels II and III, the classi-
fications for soil and rock units are not estimated but are actually measured. In the
proposed road corridor evaluated for level II and the P-line (preliminary survey line)
for level III, the specialist may also collect several soil and rock samples for testing
bearing capacities, Atterberg limits (plasticity), degradation, sand equivalency, and so
on, depending upon whether the material will be in the subgrade (undisturbed or
disturbed material underneath the roadway), base course (material placed on top of
the subgrade, sometimes referred to as road ballast), or surfacing material (aggregate
placed as surfacing on top of the base course and sometimes directly on top of the
subgrade).


Mechanics of slope failure were divided into three material types by Varnes (1978):
bedrock, debris, and earth; and six types of movement: falls, topples, slides (both
translational and rotational), lateral spreads, flows (in both bedrock and soil), and
complex (a combination of movement types). Hoek and Bray (1981) divided rock
slope failures into planar, wedge, circular, topple, and ravelling, depending on the
rock geometry and discontinuities (open or closed separations in the rock mass).
Varnes provided a rate of failure movement ranging from extremely rapid (greater
than 10 ft/sec) to extremely slow (1 ft/5 yr). Hoek and Bray did not provide a rate
of failure scale for rock slopes.


There are several soil and rock mechanics textbooks available that go into great
detail about the effect of water on the strength and movement of slopes. Both the
paper by Varnes (1978) and the textbook by Hoek and Bray (1981) provide ample
information on this topic, as does section 4 in this guide. At this point in the slope
stability assessment, the geotechnical specialist should have the location of the
surface and subsurface water and the relationship the water has to the overall...
effective stresses in the slopes being evaluated for stability. This knowledge is usually estimated for a level I stability analysis and measured for levels II and III.

15. Slope stability analysis.

The method the geotechnical specialist uses for analyzing slope stability is controlled by the scale of the project and the type of slope movement. LISA is commonly used in a level I stability analysis, but other models are also available for use in conjunction with or separately from LISA. The debris flow routing model developed by Benda and Cundy (1990) is one example. Because the infinite slope equation is the limit-equilibrium equation used in LISA, the geotechnical specialist can use this computer program for translational failures. Ristau (1988) showed that LISA can be manipulated so that rotational failures can also be evaluated if the landslide’s centroid can be located. This use of LISA, however, is unusual. In levels II and III stability analyses, the geotechnical specialist can evaluate translational, rotational, and complex landslides using SSCHFS (1988), SARA (Stability Analysis for Road Access, in process), and XSTABL (1992) computer programs. In rock slope failures, the specialist can use Hoek’s and Bray’s methods for all three levels.


Finding the factor of safety or probability of instability is only part of the slope stability analysis. The other part is to determine what effect or impact will result from a particular land modification. For a level I stability analysis, the specialist can provide probabilities of slope instability. These values can then be used in the development of various alternatives for a proposed timber harvest within the planning process required by the National Environmental Policy Act (NEPA). For a level II stability analysis, the specialist can use SSCHFS or SARA to determine how a proposed road can be constructed by evaluating cut slope angles and heights, fill slope angles and compaction values, road width, and whether or not a ditch will help keep the road prism stable. For a level III stability analysis, the specialist can use XSTABL to determine design criteria for stabilizing a slope. Section 5 provides instruction on how these tools are used and how the analysis results can be used to predict effects and impacts resulting from land modification.

17. Corrective treatment alternatives.

Once the geotechnical specialist has determined the possible consequences of slope failure, then he or she can provide some viable corrective treatment alternatives. In the level I stability analysis, these alternatives are usually tied to timber harvesting methods. For example, how will the probability of slope failure change in response to clear-cutting, shelter wood harvesting, and other partial-cutting techniques? However, this level of analysis is not limited to timber harvesting; it can also be used as part of a watershed analysis. For example, it can determine the current state of the watershed slope stability, and how it will change over a given time; if the watershed slopes have a range of instability values, how much of the material from these slopes reaches the streams or rivers on the valley floor; and if this material is a particularly high volume within the sediment budget, how this value can be lowered with certain types of corrective treatments. In the levels II and III stability analyses, the corrective treatment alternatives are not similar to the broad scale alternatives in the first level. Rather, these alternatives are tied to a specific retaining wall, support structure, or other structure designs within the road prism. Also, biotechnical
techniques within and outside of the road prism can be evaluated using these two levels of stability analyses.

18. Cost estimates.

Cost estimates are difficult to calculate for a level I stability analysis because the analysis is not site-specific. This is also somewhat true for a level II stability analysis, but the geotechnical specialist can usually provide a range of values for each design segment within a proposed road corridor. For a level III stability analysis, the specialist should be able to calculate a cost estimate for a project. Section 6 provides information on how these costs can be calculated for site-specific projects.

19. Assessment of slope stability risk.

Once the cost estimates are calculated, the geotechnical specialist can provide an assessment of the slope stability risk. Are the estimated costs for each treatment alternative effective for each associated slope stability risk? This information needs to be clear for the decision maker.


At this point in the slope stability assessment the geotechnical specialist provides recommendations in his or her report. This part of the report can easily become ponderous to read; therefore, the recommendations can be abbreviated and placed at the front of the report within a short summary, sometimes called an executive summary.


A few days after the geotechnical report has been submitted to the requester, the specialist needs to contact the requestor to find out whether the report is satisfactory. If there has been frequent communication between the specialist and the project requester, then there will be no surprises in the report product. Unfortunately, this is not always the case, and it is a painful learning experience to find out that the completed work is not what the customer originally requested. Therefore, it is important to obtain this feedback.

3C.2 Water—Key to Changes in Terrain and Limiting Use

Water is the most important part of slope stability and, therefore, understanding its effect on terrain and the limitations it places on management activities—road construction, road deconstruction or obliteration, logging, etc.—is a key part of surface investigations on soil slopes. In this portion of section 3, we provide definitions of terms used for surface and subsurface water, a review of fluid mechanics for evaluating surface and subsurface water movement, and a checklist of elements that a geotechnical specialist should use when evaluating water on a slope. The definitions and review of fluid mechanics provide a basis for understanding the use of the checklist.
3C.2.1 Definitions

Surface Water

Surface water is also called overland flow. There are two types of overland flow: Horton and saturation (Dunne and Black, 1970). Horton overland flow is the surface water that collects and runs over the land surface because the rainfall rate, \( i \), exceeds infiltration capacity, \( f \):

\[
i > f
\]

(3C.1)

Saturation overland flow is the surface water that collects and runs over the land surface because the storage, \( h_n \), and conductivity, \( k \), of the soil have been exceeded.

\[
Q = \int_{t_p}^{t_r} (i - f) \, dt > h_n
\]

(3C.2)

where:

- \( t_p \) = the time to ponding
- \( t_r \) = the rainfall duration
- \( (i - f) \, dt \) = the non-infiltrated water
- \( h_n \) = the depression storage
- \( Q \) = overland flow discharge.

Subsurface Water

Subsurface water is also called ground water. Hydrogeologists usually describe ground water using Darcy's Law, transmissivity, and storativity.

\[
Q = kIA
\]

(3C.3)

where:

- \( Q \) = the ground water discharge in \( L^3/T \)
- \( k \) = the hydraulic conductivity in \( L/T \)
- \( I \) = the hydraulic gradient in \( L/L \)
- \( A \) = the cross-sectional area of the aquifer in \( L^2 \).

\[
T = kb
\]

(3C.4)

where:

- \( T \) = the transmissivity in \( L^2T \)
- \( k \) = the hydraulic conductivity in \( L/T \)
- \( b \) = the aquifer thickness in \( L \).

\[
S = S_r b + S_y
\]

(3C.5)

where:

- \( S \) = the storativity, a unitless number
- \( S_r \) = the specific storage in \( L/L \)
- \( b \) = the aquifer thickness in \( L \)
- \( S_y \) = the specific yield in \( L^3/L^3 \).
Transmissivity and storativity are not usually used in evaluating slope stability because the geotechnical specialist is interested in evaluating the change of stresses within the slide mass, especially effective stress, $\sigma'$. This evaluation is explained in great detail in section 4E. However, before the geotechnical specialist can evaluate effective stress, an understanding is needed of ground water movement within the slide mass. This requires Darcy's Law, the continuity equation, LaPlace equation, and an evaluation of transitory surface and subsurface water movement.

### 3C.2.2 Fluid Mechanics Review

**Darcy's Law**

Darcy's Law is empirical, yet also a law (Cundy, 1992). It cannot be derived from first principles and is essentially the momentum equation for flow through a porous medium. Darcy's Law, as expressed in equation 3C.3, can be rewritten as:

$$Q = k \frac{dh}{dL} A$$

where:

- $Q$ = the ground water discharge
- $k$ = the hydraulic conductivity
- $\frac{dh}{dL}$ = the hydraulic gradient
- $A$ = the cross-sectional area.

The hydraulic gradient is defined as the change in the total water potential, $h$, to the change in the length, $L$. The total water potential is:

$$h = z + \psi$$

where:

- $z$ = the elevation head in L
- $\psi$ = the pressure potential.

The pressure potential is also known as the pressure head, $h_p$, as described in section 4.

Ground water travels through a porous medium; therefore, the flow is laminar and velocity is different than surface water velocity which has turbulent flow. Discharge for surface water is:

$$Q = VA$$

where:

- $Q$ = the discharge in L$^3$/T
- $V$ = velocity in L/T
- $A$ = the cross-sectional area in L$^2$. 
Darcian velocity (also called bulk water velocity) is:

\[ V = k \frac{dh}{dL} \]  \hspace{1cm} (3C.9)

In saturated conditions the actual velocity, \( V_a \), for ground water traveling through the porous medium is:

\[ V_a = \frac{V}{n} \]  \hspace{1cm} (3C.10)

where:

\[ V_a = \text{the actual velocity in } L/T \]
\[ V = \text{the bulk water velocity in } L/T \]
\[ n = \text{porosity, a unitless number}. \]

In unsaturated conditions the actual velocity is:

\[ V_a = \frac{V}{\theta} \]  \hspace{1cm} (3C.11)

where \( \theta = \text{the volumetric moisture content in } L^3/L^3 \).

Therefore, the geotechnical specialist needs to locate the saturated and unsaturated zones in a landslide mass when evaluating ground water velocities. This is important because a flow net depicting ground water movement cannot be constructed properly unless the velocity is evaluated properly. To show why this is the case, a short explanation of the continuity and LaPlace equations follows.

**Continuity Equation and LaPlace Equation**

If we consider a steady state condition (for a given point the ground water depth stays the same so that time is ignored) we can rewrite equations 3C.3 and 3C.9 for two-dimensional space as shown below to obtain the continuity equation (3C.13) and the LaPlace equation (3C.14).

\[ \frac{\partial Q}{\partial t} = 0 \]

\[ k_x = k \frac{\delta h}{\delta x} \]
\[ k_y = k \frac{\delta h}{\delta y} \]  \hspace{1cm} (3C.12)
In the drawing above, the flow has the dimensions of $dx$, $dy$, and $1$ (Sowers, 1979) and can be expressed as:

$$
\text{In: } V_x \, dy + V_y \, dx \\
\text{Out: } (V_x + \frac{\delta V_x}{\delta x} \, dx) \, dy + (V_y + \frac{\delta V_y}{\delta y} \, dy) \, dx
$$

Because the soil is saturated, the flow in is equal to the flow out. Collecting terms we get:

$$
\frac{\delta V_x}{\delta x} + \frac{\delta V_y}{\delta y} = 0. \tag{3C.13}
$$

Substituting the equations for velocity as shown above into equation 3C.13, we get:

$$
\frac{\delta}{\delta x} (k_x \frac{\delta h}{\delta x}) + \frac{\delta}{\delta y} (k_y \frac{\delta h}{\delta y}) = 0.
$$

For isotropic conditions $k_x = k_y$, so we can collect terms to get:

$$
\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = 0. \tag{3C.14}
$$

For anisotropic conditions ($k_x \neq k_y$), the hydraulic conductivity is:

$$
k = \sqrt{k_{\text{max}} k_{\text{min}}}. \tag{3C.15}
$$

Equation 3C.14 is the Laplace equation that describes mathematically the energy loss through a resistive medium and is used in the flow nets described in section 4E. From this description it is clear that the geotechnical specialist needs to know the nature of the ground water velocity through the landslide mass in evaluating slope stability.

**Transitory Water Movement**

Darcy’s Law and the continuity and Laplace equations are used for evaluating subsurface water flow, and section 4E gives instructions for completing the evaluation using flow nets. For surface and subsurface water flow, there are additional analyses that can be completed. Although the geotechnical specialist does not normally use these tools in slope stability analyses, they are useful in evaluating shallow slope movements, ranging from sheet erosion to debris flows; therefore, a short description of transitory water movement is presented here.

The kinematic wave approximation is one of several methods used by scientists and engineers to evaluate surface flow (Ponce, 1989). Other methods include the storage concept method and the diffusion wave approximation. The kinematic wave approximation is commonly preferred because it is easier to use than the diffusion wave approximation, and it duplicates slope conditions better than the storage concept method. Also, the kinematic wave approximation, in essence, uses steady flow for the momentum equation, and the unsteadiness is preserved in the continuity equation. The kinematic wave approximation can also be used in subsurface flow when evaluating percolation and wetting fronts (Cundy, 1992).
In evaluating transitory water movement, three variables are needed: depth of subsurface water, slope distance, and time. Subsurface water depth usually changes down the slope and over time. If, at a given point, the depth stays the same, then time can be ignored. This condition is called steady state and is not common on hillslopes. When depth changes over time for a given point, flow is called unsteady, the usual case for hillslopes. Two conditions exist for slope distance and subsurface water depth: uniform and non-uniform states. Uniform flow has a constant depth down the slope and non-uniform flow changes in depth down the slope. The schematic below illustrates these conditions.

Evaluating transitory water movement with the kinematic wave approximation requires knowing climatic conditions (e.g., rainfall intensity-duration), subsurface water conditions, and slope conditions through time. This information is usually available if the specialist is electronically monitoring an unstable slope. To complete the evaluation, the scientist or engineer can use the method of characteristics. Ponce (1989) gave a good summary of how the kinematic wave is evaluated using the method of characteristics. Fredlund and Rahardjo (1993) described in detail, using fluid and soil mechanics, how steady and unsteady states influence soil strength.
Evaluating transitory water movement is complex; however, to properly analyze a slope for stability, the geotechnical specialist needs to include surface and subsurface water movement. The kinematic wave approximation, using continuity and momentum equations, provides a mathematical method that can be a part of the slope stability analysis. A large majority of geotechnical slope stability models today do not include this transitory water movement. Current engineering hydrology, geology, and geotechnical engineering research are providing transitory water movement algorithms in limit-equilibrium slope stability models. Today, the scientists and engineers who are using electronic monitoring instruments on unstable slopes can collect the necessary data for use in these models as they become available. In the meantime, the geotechnical specialists can use their field data in available hillslope hydrology models using the kinematic wave approximation to gain a better understanding of how surface and subsurface water is moving downslope.

Checklist for analysis of transitory water movement

This checklist is a modification of Larson's original checklist compiled for a geotechnical training session in 1979.

1. Rainfall-intensity.

Field data for this element are not always available; however, the U.S. Geological Survey and Weather Bureau have data that can be used. Rainfall rate, $i$, given in several of the equations above can be determined from these data.

2. Season, location, and vegetation.

Rain-on-snow phenomena can produce unstable slopes (see section 3C.1).

3. Soil and rock classification.

By classifying the soil and rock units on the slope, the geotechnical specialist will be able to estimate the hydraulic conductivities, $k$, and infiltration capacities in the slide mass.

Several soil mechanics textbooks have $k$-values listed according to soil texture. If the soil is uniform and cohesionless the hydraulic conductivity can be calculated with the Hazen equation (Lambe and Whitman, 1969, p. 35):

$$k = 100 \left(D_{10}\right)^2$$  \hspace{1cm} (3C.16)

where $D_{10}$ is a diameter size from a sieve analysis.

Estimated $k$-values for rock units are more difficult to determine because discontinuities are more common in rock than in soil; however, rock mechanics textbooks list possible $k$-values, (e.g., Hoek and Bray, 1981, p. 18-36; Goodman, 1989). If the discontinuities are three mutually perpendicular sets and have identical smooth apertures and spacing, the $k$-value can be calculated as:

$$k = \frac{\gamma_w e^3}{6mS}$$  \hspace{1cm} (3C.17)
where:

\[ y_w = \text{the unit weight of water} \]
\[ e = \text{the discontinuity aperture} \]
\[ m = \text{the water viscosity} \]
\[ S = \text{the discontinuity spacing}. \]

Infiltration capacity, \( f \), used in equations 3C.1 and 3C.2, can be estimated after the soil is classified by referring to available tables and graphs in textbooks (e.g., Dunne and Leopold, 1978; Brooks et al., 1991).

4. Slope.

Slope measurement is needed for calculating the movement of subsurface flow.

5. Soil and water depth.

Soil depth will usually remain constant, whereas water depth may fluctuate. Changes in water depth for a given time and location on the slope are critical in evaluating transitory subsurface flow (steady, unsteady, uniform, and nonuniform states).


Concave slopes generally have more surface saturation overland flow due to subsurface flow than planar or convex slopes (see section 3C.1). Also, underlying bedrock may be controlling the slope geometry and, in turn, has a major effect on subsurface flow (e.g., structural folds and faults).

7. Slope aspect.

Slope aspect will help in evaluating rainfall intensity-duration. For example, in the Coastal Mountain Range of western Washington and Oregon, the southwest slope aspect has the highest intensity-duration rainstorms and the northeast aspect the least.

8. Roads.

Location and type of road construction will influence surface and subsurface water flow. In general, roads along or near ridge tops will have little influence. Midslope full-bench roads with high cut slopes will generally have the most influence.

3C.3 Rock Slope Structural Analysis

3C.3.1 General Introduction

This section discusses rock slope structural analysis, graphical representation, and kinematic methods for analyzing failure potential. The section draws heavily on Golder Associates (1989), Hoek and Brown (1980), Piteau and Martin (1977), and...
Watts and Associates (1986). The reader is strongly advised to obtain Golder Associates (1989) and Piteau and Martin (1977), which are in the public domain, from the Federal Highway Administration.

Structural analysis begins at regional scale and is completed with details at site-specific scale, where field data are gathered. These initial data, when reduced and analyzed, will guide the final limit-equilibrium analysis and support-system design methods covered in sections 5 and 6.

3C.3.2 Discontinuities in Rock

Rock strength is controlled by surfaces and zones of weakness within a rock mass. If rock slopes consisted of homogeneous, isotropic materials, with typical unconfined compressive strengths of 10,000 pounds per square inch or greater, it would be possible to have a 1-foot-wide column of rock over 8,000 feet tall!

Rock-slope structural analysis uses methods for categorizing, defining, and statistically analyzing structural domains, or sets of discontinuities. Rock discontinuities are defined as pre-existing surfaces of weakness along which movement, or failure, is possible. The Unified Rock Classification System defines discontinuities as two- or three-dimensional open planes of separation (OPS) or latent planes of separation (LPS).

Open planes of separation are those discontinuities that will pass air or water. Latent planes of separation may appear to be solid, but have a preferred breakage direction. These include foliations, bedding, preferred mineral alignment, discontinuities filled by secondary mineralization (hydrothermal alteration, ground water movement, or weathering), and those "healed" by stress re-application (such as volcanic or glacial loading). Other types of discontinuities which may be OPS or LPS include faults, tension cracks, heat shrinkage cracks, blasting damage, and those produced by valley stress relief from unloading during glacial retreat or stream downcutting.

3C.3.3 Methods of Data Collection

Data collection begins in the office with a thorough review of the regional technical and historical literature, followed by intensive site investigation, sample collection, and material testing.

Historical Literature Review

The regional tectonic, erosional, and depositional histories and the location of major structural features such as faults, synclines/anticlines, and lithologic rock units are important in understanding the history, origin, and processes of the project area. U.S. and State Geological Survey maps, reports, and aerial photographs are minimal requirements for historical research. If the project consists of rehabilitating aging excavated slopes, project reports written during original construction are sometimes available and very useful.

Site Reconnaissance

Initial site reconnaissance should include visual inspection of the slope for areas of previous and recent failures (wedge, plane, toppling, and rockfall) and seepage. Outcrops outside of the project limits (upslope, downslope, and along stream channels if appropriate) should be examined for rock type and major structural boundaries.
Photographs should be taken for a photo mosaic to be used with mylar overlays for later visual reference, mapping data, and project control. Vertical cross-sections should also be measured through areas that typify the slope geometry. This can be accomplished using hydraulic lift buckets or by means of rappelling. If compass readings are taken, give consideration to magnetic distortion when working around large masses of metal, such as those found in lift buckets of all-steel construction. If it is cost effective, oriented drill cores should be obtained to confirm surface mapping.

The size, frequency, and locations of these areas of failure, taken in context with the regional and local structural geology, can then be used for initial interpretation of structural domains. The structural domains control the geometric limits as a slope face map is completed, and the construction design zones are eventually assigned from this slope face map. Figures 3C.1 through 3C.4 show an example of a rock face map, typical locations of instability, remedial work, and a geologic cross-section through the rock face.

Figure 3C.1.—Rock face map developed during site reconnaissance.
Figure 3C.2.—Typical location of instability on the rock face.

Figure 3C.3.—Locations of remedial work on the rock face.
Figure 3C.4.—Cross-section B from the rock face shown in figure 3C.1.
Detailed Discontinuity Mapping

Detailed discontinuity mapping is completed after the reconnaissance phase to determine the spatial relationships between the locations and orientations of the various structural domains and their associated sets of discontinuities. These data are then reduced and statistically analyzed to determine the kinematic potential for failure in order to manage the limit-equilibrium analysis and stability design.

It is important to remember that if scaling will be done, final detailed mapping should wait until scaling is completed to ensure that a meaningful data set is used for analysis. If the detailed mapping is done before scaling, some discontinuities not exposed prior to scaling will be missed; others which may represent individual blocks may be removed, and, if included in the data, would skew the analysis. Safety is also an important reason for doing the detailed mapping after scaling, because a scaled slope will be cleaner and have less loose debris and rock that can dislodge and injure climbers.

The season in which detailed mapping is done is a safety consideration; often, rock slope failures occur during or soon after the first fall freeze or the first spring thaw. Ice expansion in discontinuities can initiate rock slope failure. Ice damming at seepage discharge points can cause the greatest case for excess pore pressure and hydrostatic forces (see section 5H for an example). Spring thaws can produce elevated pore pressures from melting snow and ice in open planes of separation.

Equipment (after Watts, 1992)

A list of field equipment necessary to complete a rock-slope mapping project includes the following:

1. FIELD GEAR

( ) Hammer, compass, hand lens, clipboard, notebook, data sheets
( ) 100-ft measuring tape, nails, chalk, flagging, paint
( ) Survey vest, backpack, rain gear, sample boxes/bags, binoculars
( ) First aid kit, climbing gear, boots, cones/signs

2. CAMERA EQUIPMENT

( ) 35 mm camera, lenses, film, tripod, camcorder, tapes, batteries

3. COMPUTER EQUIPMENT

( ) Laptop or handheld computer for data collection and reduction
( ) Handheld calculator

4. DRAFTING EQUIPMENT

( ) Graph and tracing paper, stereo nets, tape
( ) Pencils, triangles, engineer's scale
5. TESTING EQUIPMENT

( ) Handheld spring scale, bucket, water
( ) Tilt tester
( ) Schmidt hammer, pocket penetrometer, torvane, volumeter

6. REFERENCES

( ) Software manuals
( ) Hoek and Bray, FHWA manuals
( ) Site plans, photos, reports

7. COMMUNICATIONS

( ) Portable two-way radios

Structural Terms

The two most important pieces of discontinuity geometry data are the dip and dip direction. The dip is defined as the maximum inclination of the inclined plane (fall line). Dip direction is the azimuth measured in the direction of maximum dip (figure 3C.5). Along with these values, the trend, dip, and dip direction of the slope are measured with a geological compass, either a Clar, made especially for rock discontinuity mapping, or a Brunton pocket transit fitted with an azimuth scale. In order to analyze the relationships between discontinuities in a slope and the trend, or proposed alignment and dip of the slope, various methods of detailed mapping are used to obtain a statistically significant discontinuity data set.

Figure 3C.5.—Lower hemisphere projection and orientation of a discontinuity (reprinted with the permission of Golder Associates from Rock Slopes: Design, Excavation, Stabilization).
Horizontal Detailed Line Mapping

A method of horizontal detailed line mapping developed by Douglas Piteau (Piteau and Martin, 1977) in the early 1970's consisted of stretching a 100-foot measuring tape horizontally across the slope. At each point on the slope where a discontinuity intersects the tape, the position from the end of the tape is recorded, and a measurement is taken for the dip and dip direction. Other information that may influence strength parameters such as roughness, infilling material, hardness, rock type, presence of seepage, and discontinuity length is also recorded.

Most investigators use some modified form of detailed line mapping. When discontinuities measured over a length of 100 feet are projected on rectangular nets or stereo nets for analysis of failure potential, caution must be given to the "probability" of planes, wedges, or blocks of that size actually being displaced; line lengths of between 20 and 30 feet are more commonly used. When all lines have been tabulated, the discontinuities can then be plotted for graphical analysis.

As discussed in section 4D.2.5, Patton (1966) and Barton (1973) developed methods and equations to relate the angle of internal friction (φ) to the amount of roughness measured in a discontinuity. Roughness can increase the initial (or back-calculated estimate of) φ up to 15°.

Patton measured the roughness by measuring the first-order interlimb angles on the joint discontinuities. This may be difficult in the field, but, if a sample can be taken back to the office, a silhouette can be projected on the wall by using a slide projector, and the angles then can be measured at a more manageable scale. Simple tilt tests can be conducted in the field to estimate φ. (Refer to section 4D.2.5.5 for equipment and procedures.)

If a discontinuity is filled with clay, for example, undrained shear strength estimates can be made of the clay by using a torvane or pocket penetrometer. Unit weight of the clay may be calculated by obtaining a sample of known volume with an Ely volumeter, weighing the sample, and multiplying by the unit weight of water. The unit weights of rock samples can be obtained in the field by using a handheld spring scale and bucket of water to obtain the specific gravity and multiplying by the unit weight of water.

The Schmidt hammer was developed to measure the estimated unconfined compressive strength of concrete by the amount of rebound of a mechanically actuated plunger and coil spring. A chart correlating Schmidt hammer results and uniaxial compressive strength is presented in figure 4D.17.

The Unified Rock Classification System suggests using a 1-pound ball-peen hammer to estimate unconfined compressive strength directly from hammer blow impact (see appendix 3.4 for details). Portable point-load testers are also used for field tests of hardness and uniaxial strength.

Cell or Window Mapping

If the investigator observes discontinuities intersecting in the slope face 20 or 30 feet above his or her elevation, a single detailed mapping line at road level is not sufficient. This situation is more often the norm than the exception. Another
modification of Piteau's method divides the slope face into cells or windows and measures segmented lines within each cell so that all significant structures are included. This effort requires the use of mechanical lifting equipment or rock climbing skills. Mechanical self-propelled "high-lift" vehicles with reaches up to 110 feet can easily be rented. If such vehicles are used, care must be taken to have the magnetic compass sufficiently far from the bucket so that the needle is not pulled toward the metallic mass. Fiberglass buckets are more suited to this type of use. The Pacific Northwest Region now has a very effective six-person rappel mapping team.

**Vertical-Line and Outcrop Mapping**

Often, sections of a rock slope may consist of broken volcanics, or a stress-relieved mass, perhaps from valley erosion or blasting, that has more predominant individual blocks of rock having to be supported rather than removed. In measuring their orientations and size, it may be advantageous to run vertical lines from the top down in order to obtain adequate slope coverage. In this case, lines should be spaced no more than 10 feet (or the lateral reach of the climbers) apart. Often a mapping crew can follow behind a scaling crew as the scaling crew finishes its passes on the slope and is in the clear.

Two-way radio communication is vital for this type of work. The perspective one gets from a rope against a rock face is not broad enough to see overall details that may be significant from a distance. The climbing crew normally works down the slope directed by a spotter with binoculars on the ground. The ground spotter can direct the climbers to zones of discontinuities or individual blocks, and then can record the mapping information from the climber via radio as it is collected. The climber marks the area with a reference number using paint or chalk, and the spotter does the same on a mylar photo overlay with ink.

This is also an opportunity to measure the orientation of the release surfaces of blocks that have previously failed for back-calculation of typical shear strength parameters at the site.

**3C.3.4 Graphical Representation**

**Rectangular Dip Plots**

An inclined plane can be represented by a dip vector, which is composed of the direction and magnitude of the inclination. The points representing dip vectors can then be located on a rectangular grid that is divided into halves. The upper half is graduated from 0° to 90° of dip on the Y axis and 0° to 180° of direction along the X axis. The lower half is similarly divided from 90° to 0° of dip on the Y axis and 180° to 360° degrees of direction on the X axis (see figures 3C.6 and 3C.7).
Figure 3C.6.—Typical rectangular dip plot as generated by the Rockpack program. Y axis is degrees of dip; X axis is azimuth in degrees. Values in the plot are the number of discontinuities at this location.
Figure 3C.7.—Rectangular dip plot with discontinuities clustered by hand (from the computer program Rockpack).
The advantage of the rectangular dip plot is that it allows the user to quickly cluster groups of discontinuities into sets, which may be represented by a single plane of weakness common to the group, for later analysis by great circles on a stereo net (see figure 3C.7). Contouring of the discontinuity population density by computer is more efficient and accurate than are hand-calculation methods. Figure 3C.8 shows a contoured plot generated by Rockpack (1991).

Figure 3C.8.—Rectangular dip plot (from the computer program Rockpack) with the discontinuity population contoured.
Stereo Net Plots

Another method of graphically representing three-dimensional structural geology data on a two-dimensional surface is the stereo net projection. The stereo net is much more useful in analyzing structural data than is the rectangular net. Intersections of planes and the plunge and trend of the line of intersection can be obtained rapidly. Kinematic possibilities of plane, wedge, and toppling failures can also be plotted.

The stereo net is an equatorial view of a sphere, fixed in space, which is represented by a circumference (great circle plane) running through the poles. A plane that bisects the imaginary sphere at any orientation will project a semicircle (also a great circle) on the equatorial view where it passes through the lower hemisphere, if projected vertically to the equatorial plane.

Another feature that can be plotted is the point at which an imaginary line drawn perpendicular to the inclined plane, and passing through the center of the sphere, intersects the lower hemisphere. This point, when projected vertically, intersects the equatorial great circle plane and is termed the pole of the inclined plane (see figure 3C.9).

---

Figure 3C.9.—Lower hemisphere projection of pole of inclined plane (reprinted with the permission of Golder Associates from Rock Slopes: Design, Excavation, Stabilization).

There are many types of stereo nets available, but the one most commonly used in rock slope engineering is the equal-area, or Schmidt, net (see figure 3C.10).
In order to be useful, the stereo net is normally used with a mylar overlay attached to the stereo net center with a thumbtack for easy rotation. The following sequence of instructions for plotting great circles, poles, and intersections is taken, with permission, from Golder Associates (1989):

Figure 3C.10.—Schmidt net (reprinted with the permission of Golder Associates from Rock Slopes: Design, Excavation, Stabilization).
Construction of a great circle and a pole representing a plane.

Consider a plane dipping at 50° in a dip direction of 130°. The great circle and the pole representing this plane are constructed as follows:

Step 1: With the tracing paper located over the stereo net by means of the center pin, trace the circumference of the net and mark the north point. Measure off the dip direction of 130° clockwise from north and mark this position on the circumference of the net.

Step 2: Rotate the tracing about the center pin until the dip direction mark lies on the W-E axis of the net, i.e. the tracing is rotated through 40°. Measure 50° from the outer circle of the net and trace the great circle which corresponds to a plane dipping at this angle.

The position of the pole, which has a dip of (90°-50°), is found by measuring 50° from the center of the net as shown or, alternatively, 40° from the outside of the net. The pole lies on the projection of the dip direction line which, at this stage in the construction, is coincident with the W-E axis of the net.

Step 3: The tracing is now rotated back to its original position so that the north mark on the tracing coincides with the north mark on the net. The final appearance of the great circle and the pole representing a plane dipping at 50° in a dip direction of 130° is as illustrated.

Determination of the line of intersection of two planes.

Two planes, having dips of 50° and 30° and dip directions of 130° and 250° respectively, intersect. It is required to find the plunge and the trend of the line of intersection.

Step 1: One of these planes has already been described above and the great circle defining the second plane is obtained by marking the 250° dip direction on the circumference of the net, rotating the tracing until this mark lies on the W-E axis and tracing the great circle corresponding to a dip of 30°.
A useful method for portraying planes on a stereo net (as discussed for a rectangular net) is the dip vector. The dip vector is defined as a point plotted 90° from the pole, or the magnitude and direction of dip of the inclined plane (see figure 3C.11). Figures 3C.11 through 3C.15 were generated by the computer program Rockpack.
A clustering of dip vectors may represent joint sets, fault zones, or foliation with a common direction and magnitude of dip. These clusters can be contoured by population density so that a single great circle can be used to represent the cluster as a plane, or zone of weakness. This in turn can be used in a kinematic test to determine possibilities for failure (see figure 3C.12).

Figure 3C.11.—Typical computer-generated dip vector plot.
3C.3.5 Kinematic Stability Analysis

In order to identify and understand the relationships among discontinuities, potential failure planes, backslope ratios of proposed excavations, and orientations of those excavations, a static analysis can be completed on a stereo net to determine the kinematic possibility of failure of dipping planes, intersecting planes, and topples. These analyses do not include pore-water pressures or apparent cohesion, and therefore are only first approximations of stability. They provide a method of sorting out large bodies of data to determine which discontinuities, or clusters of discontinuities, could potentially fail (those dipping out of the slope at an angle steeper than \( \phi \) degrees). Among static analyses are:

1. Markland's test for plane failure (Markland, 1972). If a dip vector representing a discontinuity falls in the critical area of the stereo net, failure is possible. The critical area is defined by:

   a. The area outside of the great circle representing the orientation and dip angle of the slope face.

   b. The area inside the friction circle.
c. The area inclusive of 1a and 1b above within ±20° of the dip direction of the slope face determined by small circles (see figure 3C.13).

2. Markland's test for wedge failure (Markland, 1972). If the intersection point of two planes falls within the critical area, failure is possible (see figure 3C.14).

3. Goodman's test for toppling failure (Goodman and Bray, 1976). If a dip vector representing a discontinuity falls within 90° minus φ on the opposite side of the stereo net within ±30° of the dip direction of the slope face plus 180°, failure is possible (see figure 3C.15).
MARKLAND WEDGE ANALYSIS: C:\RKPK2-04\DATA\problem.GRT
Friction Angle = 32 degrees
Slope dip direction = 75 degrees, Dip = 85 degrees
Sets: 36 / 60 80 / 60 160 / 60 152 / 6 290 / 35

Figure 3C.14.—Markland test for wedge failure (from program Rockpack).

MARKLAND TEST PLOT: C:\RKPK2-04\DATA\sample.DAT
Friction Angle = 35 degrees
Slope dip direction = 75 degrees, Dip = 75 degrees
Number of Stations = 40

Figure 3C.15.—Markland test for plane and Goodman's test for toppling failure (from program Rockpack).
Discontinuity Significance

When the field-collected data are plotted on rectangular dip plots or stereo nets, all of the discontinuities are considered equal. All appear to be through-going, with equal shear strength characteristics, when in reality some are relatively short, have rough surfaces, and are dry. Obviously, some discontinuities or groups of discontinuities are more sensitive, or significant, to slope instability than others. Watts and West (1986) developed the discontinuity significance index (DSI) concept, which allows for ranking of discontinuities using the DSI equation:

\[
DSI = \frac{10(W\sin\psi)^{1/N}}{[K_p(W\cos\psi - U)\tan(\phi) + JRC(\log(\sigma/\sigma)) + K_m(C_mA + W\cos\psi \tan\phi_m)]^{1/N}}
\]

where:

- \( N = \text{function shape number (There are four function curves that may be used. Watts recommends using two or three to simulate actual conditions.)} \)
- \( W = \text{weight of slice} \)
- \( \psi = \text{dip angle of the discontinuity} \)
- \( K_p = \text{decimal percentage of non-intact rock along discontinuity} \)
- \( K_m = \text{decimal percentage of intact rock along discontinuity} \)
- \( U = \text{uplift pressure due to water along length of discontinuity} \)
- \( \phi = \text{friction angle along non-intact rock portions of discontinuity} \)
- \( \phi_m = \text{friction angle of intact rock in the rock mass} \)
- \( C_m = \text{cohesion value for intact rock in the rock mass} \)
- \( A = \text{length of discontinuity including both intact and non-intact portions} \)
- \( JRC = \text{joint roughness coefficient value} \)
- \( \sigma = \text{unconfined compressive strength of rock adjacent to discontinuity} \)
- \( \sigma = \text{normal stress on discontinuity} \)

The DSI number is inversely proportional to the factor of safety (FOS)—as the DSI number increases, the FOS decreases. After all discontinuities are analyzed for significance, they can be grouped by population density, or significance number, for more relevant stereo net analysis.

This method of analysis does not lend itself well to manual calculation and is more suited to computer processing.
3D. Subsurface Investigation

Mark Leverton, Engineering Geologist, Willamette National Forest

3D.1 Introduction

The success and effectiveness of any subsurface investigation depend upon many factors, which cannot all be addressed adequately to deal with the variables presented by nature, politics, money, time, and skill level. Therefore, this chapter presents some of the methods proven to be effective within typical Forest Service projects and introduces some philosophies or ways to think about the project.

Perhaps the most important concept is that sufficient pre-investigation work must be done in defining the scope and scale of the project, the probable mechanism of failure, the types of soil and rock involved, the ground water regime, and the best method to prove your hypotheses of subsurface conditions. For this reason, the concepts described earlier in this chapter involving problem definition, the scientific method, material classification, and surface investigation were presented before introducing ideas relating to subsurface investigation. Simply stated, the subsurface investigation and exploration should be used to confirm and refine your model rather than develop it.

Ideally, in addition to confirming the subsurface model developed by your surface investigation, the subsurface investigation will address the feasibility of various treatment alternatives. Therefore, during the surface investigation and job prescription you should consider what treatment options are feasible considering slope geometry, funding opportunities, and potential socioeconomic impacts. The subsurface investigation should balance material modeling with potential treatment options.

3D.2 Subsurface Investigation Tools

Often the choice of tools for use in the subsurface investigation for a particular slope stability project is a function of time, access, money, equipment availability, and risk (socioeconomic consequences of failure). A good rule of thumb when selecting tools and methods for subsurface investigations is that the degree of precision should be directly proportional to the risk (see section 61 for a full discussion of risk analysis).

A combination of exploration tools can be used to increase the degree of confidence as the site complexity and degree of risk increase. Each tool has inherent limitations, but the combination of tools should balance the limitations by setting up a system of checks and balances.

The following is a list and description of tools used effectively on Forest Service projects. This is not intended to be an exhaustive list, nor should it (in combination with the numerous tools noted in the references cited at the end of this chapter) be considered complete. Be innovative! The most important contribution that geotechnical practitioners in this organization can make to science is the development of inexpensive, innovative methods to determine subsurface conditions.
3D.2.1 Hand Tools  The most portable and available pieces of equipment for the subsurface investigation—a hand shovel, pick, sledge hammer, and rebar—also happen to be the most inexpensive. Of course, these tools provide only near-surface information unless you have a lot of time and a strong back. By digging a hole, soil samples can be obtained for field or lab classification, moisture characteristics can be determined, and an estimate can be made of in-place strength by the use of handheld equipment (torvane, pocket penetrometer, Ely volumeter, etc.). See section 4C for further discussion of these tools. Tables 3D.1 and 3D.2 give values for soil strength and soil relative density for various field test results.

---

Table 3D.1.—Soil strength related to field test results (adapted with the permission of Macmillan College Publishing Company from Introductory soil mechanics and foundations: geotechnical engineering, fourth edition, by George F. Sowers. Copyright © 1979 Macmillan College Publishing Company).

<table>
<thead>
<tr>
<th>Term</th>
<th>Unconfined Compressive Strength (After Terzaghi and Peck)</th>
<th>Field Test (after Cooling, Skempton, and Glossop)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Kips/ft²</td>
<td>kN/m²</td>
</tr>
<tr>
<td>Very soft</td>
<td>0-0.5</td>
<td>0-25</td>
</tr>
<tr>
<td>Soft</td>
<td>0.5-1</td>
<td>25-50</td>
</tr>
<tr>
<td>Firm</td>
<td>1-2</td>
<td>50-100</td>
</tr>
<tr>
<td>Stiff</td>
<td>2-3</td>
<td>100-150</td>
</tr>
<tr>
<td>Very stiff</td>
<td>3-4</td>
<td>150-200</td>
</tr>
<tr>
<td>Hard</td>
<td>4+</td>
<td>200+</td>
</tr>
</tbody>
</table>
One of the most common tools used during preliminary subsurface investigation is the hand auger. There are many varieties available, but, regardless of the variety, it should be durable, lightweight, and portable (ideally with the ability to be broken down into parts small enough to be carried easily and safely through various terrains). With experience, one can use the auger not only to retrieve remolded soil samples for material and moisture characteristics, but also to obtain an estimate of soil strength at depth by the ease of penetration, both by rotating the auger and applying moderate body weight to the auger without rotation. Remember to bring plastic bags for sample collection.

During advancement of the auger in dense material, it is possible to tighten the coupling threads to the point that they cannot be uncoupled in the field. It also is possible to strip the threads if too much force is exerted. If the auger is advanced obliquely past a rock fragment, it may be impossible to withdraw it without rotating the auger backwards (which can unscrew a coupling). Take the time to clean the auger screws by hand after each withdrawal. Banging the auger against a tree or rock can break it.

These minor drawbacks should not discourage you from using a hand auger, though. It is probably the most cost-effective tool for rapid determination of soil type and characteristics in the upper 10 feet of the soil profile.

One of the most promising innovations developed recently is the drive probe. The equipment is inexpensive and for the most part available at your local hardware store. The drive probe has been used with great success on a variety of soils projects. It yields relative strength characteristics and can serve as an observation well for ground water monitoring.
A correlation between the drive probe and standard penetration testing (which is covered in section 3D.5.6) is being sought. You are encouraged to perform a drive probe test adjacent to a standard penetration testing hole and send the hole logs of each to the Willamette National Forest, which is serving as a clearinghouse for the information. A report on the construction and use of the drive probe is included in appendix 3.6.

3D.3 Geophysical Methods

Portable geophysical equipment provides a relatively fast and inexpensive way to help determine subsurface conditions. The equipment is light and sufficiently compact to be carried easily. With experience, the procedure can be accomplished fairly quickly by two people. Data can be reduced and interpreted either in the field or in the office.

Geophysical methods can help identify areas where further exploration is necessary, or they can augment existing subsurface information. By comparing geophysical data with information derived by more direct means (such as drilling and trenching), interpretation of subsurface conditions and engineering properties can be reliably expanded in area.

There may be circumstances where geophysical methods cannot provide reliable information. In the following sections, we discuss some of the capabilities and limitations of different kinds of geophysical methods. It is important that the geotechnical specialist be familiar with the instruction manuals and technical publications on interpretation of data to recognize the more subtle limitations.

3D.3.1 Seismic Refraction

Seismic investigations depend on the propagation of waves in an elastic medium. An elastic wave or ground motion is induced by a small energy source (either a hammer and strike plate or explosives) and detected by a sensitive transducer called a geophone (USDA, 1976). The amount of time necessary for this wave to reach the geophone increases as the distance between the energy source and detector increases. Any deviation from an assumed homogeneous, isotropic condition can be interpreted to indicate changes in depth and nature of the subsurface geologic unit or units.

Two or three horizons can be detected to depths of 30 to 100 feet, depending on the equipment used and the distance between geophones (USDA, 1969). With experience and comparison with field and lab data (primarily densities), interpretations can be made for a variety of geologic conditions. Interpretation of data improves markedly with experience. However, some important limitations exist. Sufficient pre-exploration work should be done to allow for an initial subsurface interpretation prior to running seismic lines. Density or velocity of site materials must increase with depth; refractive seismographs are ineffective or of severely limited use when you try to work through asphalt, concrete, frozen soil, or any layer underlying a denser one. Also, data gathering may be impeded by weather (for example, wind, background noise, cold batteries, etc).

3D.3.2 Electrical Resistivity

Electrical resistivity investigations are based on the application of electric current to the earth through two electrodes and measuring the potential difference between two or more other electrodes. The distances between the electrodes and the measured potential difference are used to make interpretations of subsurface conditions (USDA,
This method works because earth materials are good conductors of electricity and conduct in proportion to their content of water or moisture (AASHTO, 1988) and dissolved salts or free ions. Massive rock formations and dry gravels and sand are poor conductors (with high resistivity) whereas moist soil, including clay and silt, (containing both water and dissolved ions) is a good conductor (with low resistivity).

The depth of investigation by this method can be controlled by the user. Shallow investigations are carried out by placing the electrodes relatively close together, whereas deeper investigations require increased spacing between electrodes. There is no theoretical limit to the depth of investigation, but instrument sensitivity generally limits the useful depths to 100 to 200 feet (Bison Instruments, 1969).

This investigative method has an advantage over the seismic method in that there are no masked layers because of any density or resistivity changes, and it is unaffected by frozen ground.

One drawback of the electrical resistivity method as compared to the seismic method is that it entails a more complicated determination of depths of resistivity changes. Also, natural currents (oxidized ore bodies or corroding buried metal) and artificial currents (electrical installations) may affect readings. A serious limitation of electrical resistivity investigations is the requirement that the interfaces (material boundaries) and the ground surface be parallel or nearly parallel; no dip discontinuities are allowed (USDA, 1969).

3D.4 Trenching

The construction of trenches and pits by heavy equipment is a valuable method of determining subsurface conditions, whether for exploratory purposes or for feasibility of treatment design. It allows the investigator an opportunity to see and feel (within the safety limits of OSHA and common sense) both the soil and rock material and the ground water conditions. Considerations in the use of trenching as an exploratory method include equipment type and availability, access limitations, depth requirements, and ground-disturbance constraints. Equipment such as tractors (cats), backhoes, and excavators is usually available for rental with operator from timber purchasers, road construction companies, and some Forest Service facilities. Although somewhat expensive, this method can yield a lot of information in a short amount of time.

3D.5 Drilling

Particular emphasis will be placed on drilling for a number of reasons. Although drilling is one of the most effective and widely used methods for determining subsurface conditions, it is very easy to make expensive mistakes and end up with insufficient information to effectively analyze and treat a slope stability problem. The immense number of variables presented by the site, the equipment, and the personnel requires that the geotechnical worker be well-prepared in order to direct the drilling operations before the drill arrives at the site.

The timeframe and scope of the project should be defined and an expected monetary value (EMV) or similar economic analysis should be performed to justify the use of this method of exploration. See section 6I for a discussion of EMV.
The foundation of an effective drilling project is adequate pre-investigation work. This consists of a thorough, well-marked ground survey with cross-sections through, or very near, proposed drill locations. Cross-sections and plan-view drawings should show pertinent ground features; rock and soil unit distribution, classification, and engineering characteristics; and surface-subsurface water conditions (see section 3C.1).

A determination of the equipment necessary to obtain information about subsurface conditions must be made. The most frequently used methods are core drilling, hollow stem or solid stem augering, and standard penetration testing. Because each method has limitations that depend on site materials and the type of information being sought, it is common to use a combination of these methods on any given slope stability project.

Once the geotechnical specialist has a good idea of what type of drill rig and equipment will be necessary for the project, he or she must determine how to gain access to the areas where subsurface information is necessary. NEPA documentation of some sort is necessary, possibly requiring an interdisciplinary examination of the site to define the environmental constraints under which all ground-disturbing activities will comply. Given those general guidelines, access roads for drilling equipment must be constructed with sufficient width and grade to accommodate the probable type of drill rig without adversely affecting the stability of the slope. Drilling pads also need to be constructed to allow enough working room and good footing for the crew. A 30- by 25-foot pad is generally large enough to handle a drill, portable water source, drilling equipment, and drilling inspector's area. The site geometry may force innovations and compromises. The one thing that must never be compromised is safety.

A Forest Service drill request form (figure 3D.1), or some other form of notification with the same information, must be transmitted to the drill crew or drill manager, or both, with sufficient lead time for planning and equipment gathering.
### REQUEST FOR GEOTECHNICAL EXPLORATION

<table>
<thead>
<tr>
<th>Requested by</th>
<th>Date requested</th>
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**PROJECT**

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<th>Type</th>
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<td>FRP cons - Timber</td>
</tr>
<tr>
<td>FRP precon - Timber</td>
<td>FRP cons - Timber</td>
</tr>
<tr>
<td>PC precon - Timber</td>
<td>PC cons - Timber</td>
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<tr>
<td>FRP precon - Rectea</td>
<td>FRP cons - Rectea</td>
</tr>
<tr>
<td>FRP precon - Gen Pur</td>
<td>FRP cons - Gen Pur</td>
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**LOCATION**

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**LOGISTICS**

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**EMERGENCY FACILITIES**

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<th>Location</th>
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**GOAL AND OBJECTIVE OF INVESTIGATION**

**EXPLORATION METHOD**

<table>
<thead>
<tr>
<th>Type of exploration</th>
<th>Type of material to be explored</th>
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<tbody>
<tr>
<td></td>
<td>Soil, Rock, Combination</td>
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</table>

<table>
<thead>
<tr>
<th>Number of holes</th>
<th>Last hole number</th>
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**Site Letter**

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<th>Depth</th>
<th>Instrumentation Type</th>
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**Testing procedures**

**Sampling procedure**

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<th>Type of sample container</th>
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</table>

**Number of containers needed**

<table>
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<tr>
<th>Sample containers supplied by</th>
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<tr>
<td></td>
</tr>
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</table>

**Number of core boxes needed per site**

<table>
<thead>
<tr>
<th>Samples/core delivered where</th>
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<tr>
<td></td>
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**SITE CONDITION**

<table>
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<tr>
<th>Slope at drill sites</th>
<th>Can drive to site</th>
<th>Dozer needed</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

Dozer access construction coordinated by:

<table>
<thead>
<tr>
<th>Drilling water at site</th>
<th>Will have to haul water</th>
<th>Who will haul</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
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</table>

**UTILITIES THAT MAY BE ENCOUNTERED**

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<th>Utility contact</th>
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**SURVEY CONTROL**

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<tr>
<th>Horizontal control established on the site</th>
<th>What kind of Vertical control available</th>
<th>Top Contours</th>
<th>Estimated</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Contour map accompanies this work request.

**LIMITATIONS**

<table>
<thead>
<tr>
<th>Weather restrictions</th>
<th>Site elevation</th>
<th>Environmental restrictions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**RACQFILL REQUIREMENTS**

<table>
<thead>
<tr>
<th>Follow area guidelines</th>
<th>Specific instructions</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

**FINAL DRILL HOLE MARKING**

<table>
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<tr>
<th>Pole site</th>
<th>Pole supplied by</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tag Information</th>
<th>Completed tag supplied by</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

*Figure 3D.1.—Drill request form.*
During the development of the mental model of the site, the geotechnical specialist must be considering what type of drilling equipment will be needed. Some forests or regions employ their own drill crews and equipment. If that equipment cannot accomplish the project objectives, another nearby agency may have crew and equipment available, or contracted drilling services with private sector firms may be appropriate.

If more than a few jobs are necessary, an indefinite quantity contract is useful. This type of Government contract guarantees the contractor a minimum amount of work at a given price, and the price of additional work up to a maximum amount. One advantage of this type of contract is that holes can be deepened or added until the minimum amount of work is reached. After that, additional work is paid for by work item and quantity. Another advantage is that the contract may be extended beyond 1 year without readvertisement for up to 3 years.

Specifications should be written such that inadequate equipment or services are eliminated, but not so stringently as to limit the number of prospective bidders to a few very specialized companies.

Once the bids are in, do your homework on the prospective companies. Request a listing of agencies or companies for which they have performed work (Long, 1993). As a guide, cost per foot of combined augering and coring is roughly $40 to $60, depending on mobilization miles and access.
3D.5.3 Drilling Operations

Safety is the most important aspect of any drilling operation. The job of the inspector of drilling operations is not only to create a safe working environment by access and drill pad preparation, but to plan and monitor the operations for safe procedures and work habits. The drill crew will likely do its best to accomplish the intent of your request, so your request must be reasonable. The inspector must be aware of overhead dangers and that the crew has probably worked together for some time and have well-rehearsed procedures and movements. The inspector should stay out of their way and make eye contact before entering the working area.

It is also the inspector's responsibility to develop and maintain a good working relationship with the drillers. Before the drill enters the site, it is a good idea to hold an informal meeting (jelly doughnuts are a good ice breaker) and outline what the purpose of the investigation is, what constraints they will be working under, what they will be asked to do and why, and what is expected of them and the drilling inspector. Create an environment in which the drill crew feels free to ask questions or suggest alternatives, but be clear that the inspector makes the final decisions, within reason. The drill crew is likely to have more experience in drilling operations than the inspector; listen and incorporate their ideas. A drill crew recognized for its value will produce a better work environment and product.

As operations begin, make sure that you are prepared and have all the equipment and information necessary to record information and make informed, correct decisions. Have a copy of your interpreted drawings and drafting equipment available and know the potential variables of your model. Record and serially alter your interpretation on your drawings. Once drilling begins, operations and information come quickly.

Remember to dress appropriately for the weather and working environment. A good rule of thumb is that the degree of accuracy and thoroughness in an investigation is directly proportional to personal comfort.

The following is a brief description of the most common drilling exploration methods used on slope stability projects. There is a variety of existing publications on the procedures and applications of these methods. The directions for accomplishing your task are not contained in any single publication or standard, so we suggest that you read several of these articles, as well as the AASHTO and ASTM standards, before you begin drilling operations.

3D.5.4 Core Drilling

Core drilling can be used for obtaining samples of rock units and some soil units that are either too hard to sample by soil sampling techniques or that contain a significant amount of cobble-sized or larger rock fragments. In the latter case, the material retrieved from the core barrel is probably representative of the rock fragments encountered, because finer material may be washed out by the circulation fluid.

Several methods can be used to improve the recovery of finer material. Most rotary core barrels used are the double-tube type, which is basically a single-tube barrel with a separate inner liner. The most recent advance in rotary core barrel design is the triple-tube core barrel, which adds another separate, non-rotating liner to the double-tube core barrel. This liner, which retains the sample, consists of a clear plastic solid tube or a split thin-metal liner. Each is capable of increased sample recovery in poor quality rock or semi-cemented soils, while minimizing sample handling and disturbance during removal of the core barrel. The use of "drilling
mud," with its specific gravity and viscosity greater than water, will assist in sample retention (Ruda and Basscher, 1990). Note that using these fluid additives can limit the effectiveness of ground water monitoring devices by sealing the hole with a filter (mud) cake. This filter cake can be removed by using normal mechanical well-development techniques or by the addition of polyphosphates.

Information about soil characteristics can be obtained by other exploration methods in conjunction with core drilling. This is accomplished by using drill steel of sufficient diameter to allow testing equipment or samplers through the annulus. Once a boulder or hard layer has been penetrated by core drilling, the drill steel can be uncoupled and the equipment or sampler lowered to the desired depth. One drawback to this method is that circulating fluid is required for core drilling, so information relevant to ground water is not readily apparent, and the moisture content of the retrieved sample may not be representative of subsurface conditions. A less desirable method can be used in conditions where larger diameter drill steel is unavailable. The procedure consists of simply asking the driller to stop rotation of the drill steel and to advance the hole by gentle downward pressure. The inner barrel (for double- or triple-tube drilling) must be retrieved before resumption of drilling and fluid circulation so the sample is not washed out. This method is most effective for retrieval of cohesive samples, and a relative idea of the strength of the material can be gained by the ease of penetration with constant downward pressure. The drawback of this method is that there is a risk of plugging the bit and fouling the locking mechanism of the inner barrel, either of which will require pulling the drill string for cleaning.

An alternative to core drilling using circulating fluid is the ODEX system, which uses the principles of standard percussive drilling equipment but has been modified to facilitate the installation of heavy duty, removable casing for borehole stabilization in conjunction with the drilling operation (Ruda and Basscher, 1990). The drill tool can be withdrawn into the casing. The advantage of this system is that hole advancement is more rapid than core drilling, and testing or sampling can be performed at depth without the influence of circulating fluid. The disadvantage is that limited information is available during advancement of the hole. This can be moderated, as with any percussive drilling methods, by recording advance rates and logging the chips and dust that are returned to the surface.

Another useful piece of information regarding soil characteristics that can be derived from core drilling is the permeability of the soil. This can be done in a couple of ways. The first is to watch the return of the circulating fluid to the surface. It is likely that surface circulation will be lost at some point, due either to the permeability of the soil near the surface or to interception of a more permeable layer at depth. In the latter case, knowledge of the permeability of that layer can be obtained by stopping advancement and uncoupling the steel to perform either a falling-head or constant-head test. It is useful to fill the drill steel with water before breaks, lunch, and at the end of shifts, then to measure water levels at the resumption of drilling. Keep in mind that the addition of "driller's mud" will have an effect on permeability.

**Augering**

Augering is another common method of subsurface exploration for soil projects. It allows the advancement of a hole and retrieval of remolded soil samples and indicates where water is encountered. Procedures for the use of auger systems are covered by AASHTO T 203-82 (1990) and ASTM D 1452-80 (1987); hollow-stem
augers are covered by AASHTO T 251-77 (1981). By watching the advancement of the hole at constant downward pressure (adjusted to compensate for the weight of the auger string) and listening to the sound produced, you can gain an idea of the relative strength changes with depth and the type of material present, respectively. One drawback of augering is that it does not give a precise depth at which the water or soil was encountered, particularly in cohesive soil. We suggest that while watching the rate of advance of the auger, you note a reference point on the mast relative to something other than the auger to avoid Barber Pole Syndrome (the eyes tend to follow the apparent upward movement of the auger and lose their focal point).

Hollow-stem augering is often used in conjunction with standard penetration testing. Two 18-inch drives are followed by advancing the auger to that depth. The use of hollow-stem augers has the advantages of providing a means of ensuring that the hole is vertical, reducing skin friction on the drill steel, and providing greater reliability that the soil sample in the split barrel is from the desired depth. Augers also can pass through obstructions. Rubber O-rings can be installed at the connection of individual flights to reduce water inflow above the bottom of the hole. The development of removable auger bits, retrievable through the annulus of the augers, has greatly improved the efficiency of operations.

3D.5.6 Standard Penetration Testing (SPT)

SPT is one of the oldest and most widely used in-place tests of subsurface soil conditions. Procedures for penetration testing and split-barrel sampling of soils are covered in AASHTO T 206-87 and ASTM D 1586-84 as well as a wide variety of other reference materials.

Effective SPT is much more complicated than the procedures in the references portray. Therefore, it is especially important that the geotechnical specialist has all of the equipment and pre-drilling interpretative work organized and available before operations begin. At a minimum, the specialist will observe and document drilling operations, classify material, select and prepare samples, prepare field logs, and modify pre-drilling interpretation on drawings. Additional duties often include removal of samples and cleaning of the split spoon.

Recommended equipment includes: a water-resistant field book, several pencils, rags, a 5-gallon bucket of water for cleaning hands and equipment, knife, hand lens, permanent markers (felt pens or grease pencils), baby food jars, sample containers, lab tags, propane stove, classification charts, blow count conversion charts, water level measuring device, and surveying equipment.

It is valuable to observe the penetration distance with each blow. This is particularly true when rock fragments, which may result in blow counts not representative of subsurface conditions.

It is best to perform drilling operations during a season providing the most information, such as when the subsurface water system has recharged. This allows observation of subsurface conditions more representative of failure conditions; it also provides better reliability in setting up a water monitoring system. Unfortunately, access may not be practical due to snow or soil strengths that are too low to support drilling equipment.
An important observation to be made during drilling is the character, location, and behavior of water. Note whether the water content of the soil gradually increases or is in a particular zone or discontinuity. Upon encountering water, it is often valuable to stop operations for a few minutes to observe whether the water level rises or is static. Also, measure and record the water level before breaks, lunch, at end of shift, and again at the resumption of hole advancement. Samples collected for moisture content should be placed in a tight, waterproof container as soon as possible after removal from the split barrel.

Thin-walled tubes, referred to as Shelby tubes, can be used to obtain relatively undisturbed samples or to provide smooth-walled holes. Undisturbed samples can be used for direct shear or triaxial shear testing. The smooth-walled hole left after the extraction of the tube can be used for in-place testing, such as with an Iowa borehole shear device. Further discussion of these and other tests is included in section 4C.

Procedures for obtaining Shelby tube samples are covered in AASHTO T 207-87. Field experience with Shelby tube sampling has shown that: (1) This method works best in silt, clay, and fine sand; (2) The coarser the material, the more likely that problems such as loss of sample, bridging, and buckling of the tube will occur; (3) The more consolidated the soil, the more difficult it will be to advance the Shelby tube, with a corresponding increase in the risk of buckling; and (4) Murphy's Law states that if there is only one cobble-sized rock fragment in a fine-grained soil, it will be where you end up trying to push a Shelby tube.

During drilling operations, observations, and subsequent monitoring, the extent and nature of the ground water regime should be determined. One of the most important observations to be made is whether individual ground water surfaces are perched, confined, or phreatic. The significance of these measurements and their effects on slope stability will be explained and discussed in section 4E.

Many references exist for equipment and procedures for installation of ground water monitoring devices. A few of the more useful ones are included in the reference list at the end of this section. Familiarization with several references, as well as pertinent state regulations regarding monitoring holes, is recommended.

Some guiding principles for ground water monitoring device installation are: (1) Use clean sand for backfill to decrease the chance of bridging; (2) Wash backfill down the hole with plenty of water; (3) Take frequent depth measurements; (4) Allow time for sand or bentonite pellets to settle out; and (5) Consider coated or specialized bentonite pellets for deeper installations to reduce the chance of pellets adhering to one another during descent.

Upon completion of installation, clearly label the hole, measure the installed depth, and check for deflections in the casing. This can be done by lowering a length of pipe to the bottom of the hole. A 1/2-inch- by 4-foot-length of conduit tied to a measuring tape is effective. A shorter (6-inch) conduit works well for measuring water levels. A distinctive sound is heard when the annulus of the conduit encounters the water surface. It is also useful to measure the length of pipe sticking out of the ground. On several occasions, slope movement without pipe shear has resulted in changes in length above the surface.
To check whether the ground water monitoring device is functioning properly, fill the pipe with water periodically to see that the measured depth can be repeated. Remember, it may take some time for the seals to become functional if bentonite pellets or chunks are used.

See section 4E for a complete discussion of piezometer arrangements for phreatic and piezometric surfaces, remote electronic monitoring devices, and developing the model from the data; and section 6D.2 for a discussion and examples of methods using resistivity, dye tracing, permeability testing, and modeling.

3D.7
Suggestions for Completion of Drill Holes for Geotechnical Monitoring

3D.7.1 Purpose

The primary purpose for drilling the hole in a geotechnical investigation is to recover the subsurface data necessary for design. However, because certain design variables, such as free ground water, are time dependent, the measurements made at the time of drilling may not be representative for the critical state design. In addition, the effect of construction on the site conditions can frequently only be determined through postdrilling monitoring, which can be facilitated by using the subsurface access available through the drill hole. Proper casing, backfilling, and sealing of the drill hole allow for the acquisition of this postdrilling information (which may be more valuable than that gained during the initial investigation). Postdrilling monitoring can help determine or verify:

1. Values for design variables, such as free ground water depth.

2. The effects of construction on site conditions:
   - How effective drainage systems are in lowering the free ground water surface.
   - How effective slope stabilization techniques are in reducing or stopping landslide movement.
   - The amount of settlement within a large fill or the amount of fill subsidence due to consolidation of the soil beneath. Also, what effect this combined settlement and subsidence has on the road, landslide, and such.

3D.7.2 Typical Monitoring Instruments

Many instruments are available for providing geotechnical data on soil moisture, free ground water, displacement, and load. The four most applicable to Forest Service road construction, and the monitoring data they provide are:

1. Recording piezometer: Used in observation wells to monitor the fluctuations of free ground water.
(2) Vertical inclinometer: Used for measurement of lateral displacement of specially designed grooved casings.

(3) Horizontal inclinometer: Used for measurement of vertical displacement of specially designed grooved casings. This requires a special sonde and the installation of a dead-end pulley to pull the sonde into the hole.

(4) Settlement indicator: Used to measure the amount of settlement due to consolidation of the material within a fill or the amount of subsidence under a fill due to consolidation or displacement of the underlying in-situ soil material.

The following installation and backfill recommendations are for casings to accommodate these four monitoring devices.

3D.7.3 Casing Installation and Backfill

Regardless of the intended function of the casing, the relationship between its outside diameter (o.d.)—or the casing coupling o.d., if used—and the hole diameter should allow sufficient annulus between the hole and casing for effective placement of the backfill material. The hole diameter should be at least 1 inch larger than the casing o.d. (or coupling o.d., if used) to prevent bridging of the backfill material during placement. The maximum size of backfill material should be about one-quarter of that, or 1/4-inch. This allows the installation of 1½-inch flush-coupled PVC pipe (o.d. = 1.9 inches) or 1-inch PVC pipe with couplings (coupling o.d. = 1.7 inches) in a 3-inch drill hole. For example, the 1/4-inch backfill could consist of concrete sand (no.100–no. 4) for drainage aggregate and no. 4 mesh or powder bentonite (3/8-inch pellets might work if properly tamped) for sealing backfill.

Recording Piezometer

It is important to distinguish between confined and unconfined aquifers during drilling and to determine the depth limits of each. This is necessary for proper sloting and sealing of the drill hole to access and isolate the aquifer (figure 3D.2). The deeper confined aquifer can be isolated and monitored only if: (1) the casing is slotted and backfilled with drainage aggregate through the aquifer thickness, (2) the casing is unslotted from there to the ground surface, and (3) the backfill is sealed within the confining layer that confines the aquifer. Installation of the casing to access the upper unconfined aquifer is less critical because it extends to the ground surface; the casing can be: (1) slotted to within 18 inches of the ground surface and backfilled with drainage aggregate and (2) unslotted and sealed with bentonite (to prevent surface water from infiltrating the backfill) from there to the ground surface.

Flush-coupled 1 1/4- or 1 1/2-inch PVC pipe, such as that used for horizontal drains, makes excellent observation well casing. It comes in 10-foot unslotted or preslotted (with specified slot width) lengths; however, because the slotted and unslotted couplings must be placed at the appropriate depths in the field, preslotting is sometimes of little advantage. This casing is also relatively expensive. For most installations, standard 1-inch PVC pipe, available at hardware stores in 20-foot lengths, is more economical and perfectly satisfactory. At the site, slots can be made with a hacksaw at the aquifer depth about 4 inches apart and one-quarter of the way through the pipe diameter.
The 1.7-inch-o.d. couplings require a 3-inch drill hole for adequate backfill around the coupling. A minimum 1-inch-inside-diameter (i.d.) casing is recommended to allow installation of the piezometer sensor (which remains in the well) with adequate space remaining for the sensor of a resistivity meter during periodic manual checks. All pertinent data—such as casing size and type, slotting depth, drainage aggregate type and depth and soil material type and depth—should be recorded on the drilling log or in an installation notebook in the project file.

Electric piezometers and low cost data recorders provide the opportunity to use multiple isolated piezometers in sealed drill holes to measure pore water pressures without the volume change and lag time associated with open standpipes. Multiple piezometer installations, similar to C in figure 3D.2, are better able to detect multiple ground water zones than with open standpipes.

Figure 3D.2.—Casings installed for recording piezometers in confined and unconfined aquifers.

Subsurface conditions are seldom as simple as illustrated in figure 3D.2. Many, if not most, large landslides are a complex of water-bearing and water-restricting zones created during events leading to the current condition. Two brief case histories are
offered to illustrate the need to understand and instrument the ground water conditions at the site.

Case History: Thomas Creek Landslide, Fremont NF, Oregon (Steward, 1994)

A 200-foot section of the road accessing the Mill Creek drainage dropped about 6 inches in late winter (about 1973). The landslide area was cleared and several holes were drilled with a 4-inch o.d. auger. Perforated plastic pipes were installed in all the holes similar to A in figure 3D.2. A 30-day strip chart recorder with a water level float was installed on one hole near the center of the slide area. The maximum water level came to within 12 feet of the ground surface. A stability analysis was performed using site geometry, measured ground water levels, and back-calculated strength parameters. The analysis indicated removal of landslide material above the road would increase the stability of the slide. Removal of 10–15 feet of material resulted in the road dropping about 2 feet and moving toward Mill Creek. Ground movement was evident in several directions, but none in the direction expected from the surface geometry and the stability analysis. Ground surface cracks indicated up to seven interconnected sliding areas existed within the main slide area.

We then suspected a deep, confined aquifer with high water pressure was involved. The area was redrilled using a wireline core drill with soil samples taken through the core barrel. A new drill hole located near the center of the slide (near the previously auger hole with the water level recorder) told the rest of the story. At about the 50-foot depth during drilling, the drill rod dropped about 5 feet with no effort, and entered a black organic layer followed by a clayey layer. A confined aquifer piezometer (B in figure 3D.2) was installed. The water level rose to the ground surface, a full 12 feet higher than in the nearby auger hole. A permeable rocky zone at 12-foot depth had allowed ground water to escape from the original holes. Re-analysis of the landslide with the new ground water information confirmed that we would destabilize the slide if we removed landslide material above the road.

The slide was stabilized by drilling 200- to 275-foot-long horizontal drains into the deeper water bearing zone. A 12-foot deep geotextile lined aggregate trench drain was installed in the road ditch line to remove near surface water.

Case History: Switch Creek (formerly Austin Point) Landslide, Mt. Hood NF, Oregon (Steward, 1994)

This very large landslide became active about 1974 and threatened to close the Clackamas River Road, a two-lane paved main access road, and disrupt flow of the Clackamas River. The Switch Creek landslide was renamed after 1976 when a large landslide occurred closer to the local landmark, Austin Point.

A geotechnical consultant was retained to investigate the landslide and to develop correction alternatives. Based on what could be measured at the site, the estimated cost of the corrections ranged from $3 to $7 million. The consultant’s report indicated that the ground water level or pore water pressure was likely higher than had been measured in order to cause the slide to move as it was moving. One key indicator was water seepage noted above the road level, but not picked up in the
Another key indicator was the observation noted in the drill log that the water level in one of the drill holes increased overnight when the hole was partially completed. The water level measured in this hole remained very deep after the hole was completed. These observations indicated one or more perched or confined water levels existed in the landslide.

Several new drill holes were drilled by the Forest Service with multiple pneumatic piezometers installed in each hole. These piezometers verified the consultant's observation about the ground water. Multiple ground water zones were measured and the response to rainfall indicated some ground water zones interconnected and some were independent. Pore water pressures equivalent to water near the ground surface were measured deep in the landslide. The information gained from the multiple piezometers allowed the landslide to be stabilized for about $1.1 million in 1976. Stabilization measures included a deep key trench, massive rock buttress with geotextile filter, horizontal drains, and regrading.

The standpipe piezometer installations in figure 3D.2 (A and B) are relatively easy to install and permit measuring the water level by hand methods or using electric piezometers with a data recorder. These methods are adequate, unless the landslide and related ground water distributions and pore water pressures are complex. Multiple piezometers can be more difficult to install than the standpipe piezometers, and if damaged, cannot be replaced without drilling a new hole. However, two to four piezometers can be installed and isolated from each other in one drill hole, depending on the diameter of the drill hole, the drilling equipment used, and the skill of the personnel. Properly installed multiple piezometers can save drilling costs and improve information collection for complex ground water conditions. Electric piezometers are compatible with relatively low cost data recorders.

**Vertical Inclinometer**

This instrument requires a specially designed grooved casing to maintain proper orientation of the sonde. There are two sets of grooves: one set oriented in the anticipated direction of lateral displacement, and the other at right angles to that direction. Manufacturers of the instrument and grooved casing provide excellent instructions for installation and backfill. Their instructions should be followed and will not be duplicated here.

The size of the grooved casing is determined by the instrument sonde. The two most commonly used grooved casings are flush-coupled 1.9-inch-o.d. PVC and 3.4-inch-o.d. aluminum with external couplings. The minimum drill hole sizes should be 3 and 4-1/2 inches, respectively. For these casings, concrete sand (no. 100–no. 4) or pea gravel (fine concrete aggregate, no. 4–3/4-inch) are commonly used for large diameter borings. For small diameter borings, concrete sand screened through a no. 16 sieve (no. 100–no. 16) is recommended (specification concrete sand is about 65 percent passing the no. 16 sieve). The sand should be densified at the time of placement to prevent void spaces and later settlement of the backfill that could give false casing deflection readings. Densification is best accomplished by vibration of the casing. Some manufacturers make an air-driven vibrator that operates inside the casing for this purpose. All pertinent data for the casing should be recorded. Initial readings of displacement should be taken at completion of the casing installation to be the standard of comparison for all subsequent readings.
Horizontal Inclinometer

This instrument is similar to the vertical inclinometer but requires a special sonde and a dead-end cap with a pulley for pulling the sonde into the casing (figure 3D.3). Plastic “poke holes” (1/2-inch o.d. plastic pipe) have been used to push the horizontal inclinometer into short holes less than 25 feet long) in lieu of the pulley and cable assembly. The casing is usually not placed in a drill hole but installed during construction at various lifts in an embankment. However, in order to distinguish between settlement deflection and subsidence reflection, some drilling is required through the in-situ soil under the embankment, as shown in the figure. In this zone, the annulus between the drill hole and casing must be backfilled by pressure grouting with a lean cement or clay cement grout. The 3.4-inch-o.d. aluminum casing with external couplings is typically used. An additional 1/2-inch PVC conduit is required for the permanently installed pull line. The grout injection may force some grout into the aluminum casing at the couplings, the end cap, or the PVC conduit. These should be flushed out with clean water before the grout has hardened. The casing grooves should also be cleaned with a wire brush during the flushing operation.
Figure 3D.3.—Horizontal inclinometer casing installed in road fill.
The casing is installed with one set of grooves horizontal and the other vertical. The drilled-in portion of the casing and the pull line conduit should be capped and protected from construction damage just below the ground line. The horizontal and vertical location of the ground surface of the cap location must be referenced off the construction area for later relocation and as a standard for settlement measurement. After the fill has been constructed and compacted to an elevation about 6 inches above the drilled-in casing, the casing can be extended across the fill. The cap is relocated and a trench excavated to conform to the elevation and direction of the drilled-in portion of the casing. The pipe and pull line are then extended to the monitoring location at the fill slope. Backfill of concrete sand is tamped into the trench. Additional concrete sand cover, 6 inches in depth, is placed over the pipe and compacted with a mechanical tamper to protect the casing before the next lift of fill is placed. All pertinent casing data are recorded. Initial readings are taken on the casing at the completion of the drilled-in portion, at the completion of extension across the fill and backfill operation, and at completion of each additional 5 feet of fill.

Settlement Indicator

Most settlement measuring devices are installed during construction of the fill and do not require drilling. However, in order to determine the amount of subsidence of the in-situ soil under the fill it is necessary to drill through the foundation soil. An excellent settlement measurement device utilizes the 3.4-inch-o.d. aluminum vertical inclinometer casing with external telescoping couplings. Lateral displacement can still be measured with the vertical inclinometer, but, in addition, the relative movement between adjacent 5-foot casing lengths can be measured in the coupling space (figure 3D.4).
Figure 3D.4.—Vertical inclinometer casing with telescoping couplings for settlement and subsidence measurement.
A 6-inch settlement flange is attached to each length of casing above ground surface–fill contact in each 5-foot lift of fill. The couplings are riveted only to the lower casing to allow the upper casing section to telescope into the couplings where the displacement can be measured. Installation below the fill should be as described for the vertical inclinometer except that (1) telescoping couplings are used, (2) the casing is grouted or otherwise anchored at the bottom of the hole, and (3) a settlement flange is installed at the ground surface. Metal settlement plates or rings can be installed on plastic casing loosely, without attachment) during placement of the fill and located after construction using an inductance device lowered into the hole. The horizontal and vertical location of the ground surface top-of-casing must be referenced off the construction area for later relocation and as a standard for settlement measurement.

To minimize interference with the construction activity, (1) the casing is installed to near ground surface, (2) the casing is capped for protection until the 5-foot lift of fill is placed over the location, (3) a pit is excavated to the capped casing, (4) the next length of 5-foot casing with a telescoping coupling and settlement flange is installed and the pit backfilled and compacted, and (5) settlement readings are taken on all installed sections. This must be repeated for each 5-foot lift of fill. All pertinent data must be recorded and initial settlement readings taken prior to fill placement and with the completion of each 5-foot lift of fill.

There is an inexpensive alternative that allows the measurement of total settlement within the fill and of total subsidence under the fill. It requires permanently anchored, shielded wire line cable extensometers extended up through the fill in 5-foot lifts with 1/2-inch PVC conduit, similar to the vertical inclinometer casing without the telescoping couplings and settlement flanges (figure 3D.5). Two cables are necessary: one anchored at the base of fill and one below the foundation soil. The cable length to the anchor must be premeasured and permanently marked in the shielding at 5-foot intervals. The horizontal and vertical location of the top-of-ground surface must be referenced off the construction area for later relocations and as a standard for the settlement measurement. The exact length of 1/2-inch PVC pipe must also be measured at each lift to provide a reference for the relative difference between the conduit length and wire line length. The conduit should not extend to the anchor but allow about a 2-foot (or whatever the anticipated range of expected settlement is) gap so that the conduit has room for vertical displacement as the fill settles. Backfilling of the 1/2-inch conduit is not as critical as for the inclinometer casing because lateral displacement will not be measured.
Figure 3D.5—Simple wire line extensometers for measurement of total fill settlement and subsidence.
There are probably as many methods of portraying subsurface information on drilling logs as there are geotechnical practitioners. Numerous software packages that produce high quality products are available. Many forms have been produced by various agencies. But neither the software nor the form guarantees a useful log; that responsibility lies with the person who inspects the drilling operation and produces the final drill log.

The final drill log can be thought of as the final representation of the drilling effort and the site materials. A lot of physical and mental effort goes into each hole, and the log should reflect that. When producing a drill log, consider that another person who uses it may not have access to your memory. As with many tasks where documentation of effort is critical, the following saying is good to remember: "If you did something and you didn’t write it down, you didn’t do it; if you did something and wrote it down but can’t find it, you didn’t do it." (Williamson, personal communication). Make sure that your documentation process is thorough and understandable to others. You may not be the last person working on the project.

It is not necessary to repeat all of the information from each drill hole field log on the final log as long as the field logs are legible, understandable, and accessible, but the final log must contain enough substance to adequately portray the information and interpretations derived from the drilling operation. It is best to use a blend of graphical representation and text description. Graphics can be used to portray boundary conditions (classification, origin and strength changes, water-bearing zones, failure surfaces, etc.). Text can be used to describe material conditions, operational information, and such. It is important that the log have a graduated scale for depth.
3E. Geotechnical Monitoring

Richard VanDyke, Geotechnical Engineer, Siskiyou National Forest

3E.1 Importance of Monitoring

Monitoring can provide valuable information on incipient as well as active landslides. Monitoring of potential slide areas can serve both as a warning system and as a method of gathering important data for use in the analysis of possible repairs. The monitoring of active landslides is one of the few methods of gathering accurate data that can be helpful in determining the causes of a stability problem and in analyzing and designing a solution to stabilizing the landslide. Monitoring can also provide the necessary information to determine whether a constructed repair is working effectively and in accordance with the design requirements. Typical information that can be obtained from monitoring is:

1. **Depth and shape of the sliding surface.**
2. **Vertical and horizontal limits of the landslide.**
3. **Rate of movement of the landslide.**
4. **Ground water levels or pore pressures.**
5. **Relationship of precipitation, ground water, and movement.**
6. **Effects of corrective measures regarding some or all of the above.**

Without the information acquired through some type of monitoring system, many of the variables that have to be identified during a stability analysis remain unknown. As a result, the accuracy with which the slide can be modeled decreases, lowering the designer’s confidence in the reliability of the analysis and the resulting recommendations. This will generally result in more conservative designs and thus higher repair costs.

3E.2 Instrumentation Planning

Determining the amount and type of monitoring best suited for a particular slide requires adequate planning and preparation. The planning must involve forming a hypothesis as to the causes of the landslide, as well as estimating the surface limits and depth of the slide (see section 3A and appendix 3.1 on the scientific method). With this hypothesis and the knowledge of possible repair options, a monitoring system can be designed for a particular site. Generally, the instrumentation used should be sufficient to define the magnitude, rate, and distribution of the movement and include some method to collect ground water readings (see section 4E.6). If the slide is relatively small and the depth and limits can be determined from surface features, monitoring may be neither economical nor necessary to gather the information required to perform an analysis. Likewise, if it is critical that the repairs be
accomplished within a very short time, it may not be practical to install complex or sophisticated monitoring.

If the landslide is large and complex, or if the depth of the failure and ground water information cannot be inferred from mapping surface features, then monitoring with inclinometers, standpipes, piezometer, or similar devices may be required to collect adequate data. These types of monitoring systems are generally installed during the drill portion of the subsurface investigation. The cost of installing monitoring when it is done in conjunction with the subsurface investigation is relatively low, and it is a good practice to install at least a slotted PVC pipe in any drill holes that have water level information (see section 3D.6). Accurate surface measurements of scarp height, crack location, and so forth, should be collected and referenced during the survey for use as base data for monitoring changes.

3E.3 Types of Monitoring

In most landslides, the two critical areas that need to be monitored are the slope movement (magnitude, rate, time movement occurs, etc.), and the changes in ground water levels or pore pressures. The instrumentation available for each of these types of monitoring ranges from simple manual systems to sophisticated electrical devices. The particular equipment selected depends upon the size and complexity of the slide, the overall significance of the site, the cost of the various systems, and the time associated with gathering the data. (For exact descriptions of how to set up and conduct the following types of monitoring, see Wilson and Mikkelsen, 1978.)

3E.3.1. Survey

Ground surveys are essential to the investigation of all sizes of slides, although the degree of accuracy and sophistication may vary. It is most convenient while the site is being surveyed to locate its boundaries, and such, to install some surface controls that can later be checked to monitor surface displacements.

Standard surveying equipment can be used to install these monitoring systems. The desired accuracy with which they are placed will depend upon the anticipated magnitudes of movement. The survey monitoring should start with the location of several benchmarks well off the active area. The subsequent monitoring can consist of survey lines, grids, targets, and elevations to monitor both horizontal and vertical displacements.

If definite boundaries and scarps have been identified, another useful and inexpensive monitoring technique is the use of stake arrays placed across scarps at the limits of the active areas. To help locate the limits, you must sometimes look for such subtle indications as disturbed vegetation, distortion of shrubs and trees, stretched roots, and displaced rocks or soil. Stake arrays require only an accurate tape measure that will not stretch and some type of permanent markers, such as metal or plastic stakes or pipes. By installing two stakes off the active area and two stakes on the slide mass and taking measurements from each stake to the other three, the magnitude and direction of movement can be determined from subsequent readings.

3E.3.2 Photogrammetry

Photogrammetric methods can be used on various sizes of slides to monitor slide changes. For large slides, aerial photography will help delineate the boundaries and determine the relationship of the area of concern to the surrounding geography and geology. Subsequent flights can be made to reveal any significant changes. Varying
amounts of detail and precision can be obtained depending upon the elevation of the flight, equipment, and such. If stereo pairs are taken, measurements of surface irregularities and changes can be obtained with sophisticated electronic stereo plotters. With good control, some of these methods are accurate to within less than 1 foot. These methods are generally expensive and use specialized equipment and thus are practical only for large critical sites. However, on-the-ground photography with either still or video cameras will also show changes that occur over time. Using this type of monitoring requires that specific photo points be located well off the active area and that both the angles of the photographs and the lighting be duplicated in subsequent photos.

3E.3.3 Extensometers

Extensometers are another method of measuring the displacement that takes place between two areas of concern. These areas can either be on the surface, such as across a scarp, or between contacts below the surface. Extensometers can either be simple homemade devices read manually on a routine basis or one of several sophisticated systems that take readings automatically at regular intervals. For the information collected from extensometers to be useful, both the magnitude of movement and the exact time the movement was initiated in relation to other factors such as precipitation or ground water levels need to be known. For this reason, the type of extensometers that have to be read manually are of little use other than for academic information or to verify that movement is taking place. It is not realistic to obtain manual readings frequently enough to provide useful data for the analysis process. On the other hand, the automatic recording equipment now available (for example, the continuously recording extensometer developed by Rod Prellwitz; see section 3D.6) can record changes in displacement on as short as 15-minute intervals for several months. The equipment is battery-operated, lightweight, reusable, and easily installed. The information can be downloaded into a personal computer and the resulting displacements plotted on a graph of displacement versus time. When set up in conjunction with ground water recording equipment, this provides invaluable data to be used in the analysis of the stability of the slide. Of course, not all slides demand such sophisticated monitoring, but it is available for complex or critical sites.

3E.3.4 Inclinometers

In order to determine the depth at which movement is occurring on a slide, some type of subsurface monitoring is required. These devices and techniques can also be either simple or sophisticated. A simple technique may consist of manually checking how far various lengths of rods can be lowered down plastic pipes installed in investigation drill holes. A slightly more sophisticated method to determine whether movement is occurring at more than one level can be set up by leaving sections of rebar attached to wire rope at various levels in the pipe and pulling them up to check for displacements in the pipe. If the magnitude of movement is relatively large then either of these methods works well. However, if very small movements are expected, then more sophisticated and more expensive equipment must be used. The standard method involves the installation of grooved plastic pipe down some type of drill hole. With the use of an inclinometer, which measures angles of deflection between various points at specified depths along the plastic pipe, the location as well as the amount and direction of movement can be determined. With the latest equipment, this information can be stored on portable recorders and downloaded into a personal computer and graphs plotted showing the location and magnitude of displacements. Because of the length of the inclinometer device that has to be
lowered down the pipe, this method is not useful once displacement occurs that prevents the measuring device from being lowered below the slip surface. Thus, for very rapidly moving slides, the reading must be taken at short intervals so subsequent readings can be compared. If only one set of readings is collected before the pipe shears off or displacements prevent the reader from being lowered down the pipe, then you might as well have used the simple, less expensive method.

Because ground water and pore pressures play such an important role in the overall stability of most slopes, it is very important that some type of monitoring be established to gather data regarding precipitation, surface water, and ground water or pore pressures. The monitoring of precipitation is mainly academic, but if enough information can be collected to correlate it with slide activity, then potentially useful predictions about future movements can be made. Rain gages can be installed at the site and equipped with continuously reading data recorders to obtain intensity and duration of rainfall.

More directly applicable in the stability analysis is knowledge of changes in the ground water or pore pressures. The systems used to monitor ground water, like those to monitor movement, can be either simple or elaborate depending upon the complexity and seriousness of the slide. The simplest and most economical method is to install plastic pipes in the holes used for subsurface investigation. The pipes can be slotted with a hand saw and the zones to be monitored can be isolated by placing bentonite clay in the drill hole when backfilling it. After the water levels have stabilized, readings should be taken as frequently as necessary to collect the information to be used in the stability analysis.

Past experience has indicated that in many situations the response time of the ground water to precipitation can be very rapid, and the overall pressure changes that various soil layers are exposed to due to extreme ground water fluctuations can be quite large. Because of the possibility of missing the critical measurements, hand monitoring has proven to be unreliable for collecting information on critical sites. Continuous monitoring systems developed by Rod Prellwitz have proven to be reliable, accurate, and easy to install. They use a sensitive pressure transducer, an electronic timing device, a voltage regulator, and a recording device. The same recording equipment may be used for the extensometer readings and precipitation monitoring, with one system recording both. This allows synchronization and correlation of readings, which is valuable for determining the critical relationships that occur and which must be used in stability analysis and the repair options.

If an effective monitoring system has been established and followed, the assumptions are clearer and the necessary work is easier in doing the actual stability analysis. This will result in a more accurate analysis and a more economical solution than if the values had to be conservatively estimated, or if the wrong conclusions were assumed. In most cases, monitoring can be justified from both safety and economical standpoints.

3E.3.5
Precipitation,
Ground Water, and
Pore Pressures
References


References


**Level I Stability Analysis Ver. 2.0 (LISA) 1991.** Moscow, ID: U.S. Department of Agriculture, Forest Service, Intermountain Research Station, Engineering Technology.


APPENDIX 3.1
3.1 Review of the Scientific Method

Introduction

The following review is from a course developed and taught by Professor Ford at the University of Washington (Ford, 1991). This is an abbreviated version and does not cover all the fine points; however, the references at the end of the review will guide the reader to a comprehensive understanding of the scientific method.

Science is commonly believed to be a process of increasing our certainty. Within a theory there are ideas of varying degrees of certainty. The enigma of science is that we start from uncertainty and have to use our subjective thoughts and ideas, our imagination, to initiate the process.

Purpose

Definition: The purpose of the scientific method is to place the subjective process of developing new ideas into a logical framework of challenge and questioning. This results in the development of objective knowledge.

The scientific method does not deny the use of subjective thoughts. But its principal task is to ensure that new ideas find their proper place, and so the whole theory is developed. Ford considers all scientific research to have the purpose of developing objective knowledge. The method by which this is done is the same for all disciplines. It involves constant challenge and questioning.

Objective Knowledge

Definition: Objective knowledge is knowledge that has been subject to scrutiny and debate among scientists. It has been tested by having been "objected" to. It is independent of a single person, so it is not subjective knowledge. However, in contrast to some common uses, objective knowledge is not absolute or permanent. It changes as scientific investigation continues.

The definitions of scientific method and objective knowledge are linked. They do not exist in isolation from each other. This linkage between the state or condition of knowledge and the processes that produce it is seen as we analyze the scientific method in more detail.

A Model for the Process of the Scientific Method

We can describe a model of the scientific method common to all scientific disciplines. The model defines a process of progression toward increased objective knowledge. The scientific method involves the use of four thought processes: imagination, deduction, investigation, and comparison, used to develop ideas among three states: axioms, postulates, and data (figure 3.1.1).
Definitions

A body of scientific theory is composed of propositions motivated by a set of questions.

A concept has two properties. It is an abstraction that has an assigned definition.

A proposition is a set of concepts linked in a logical way that forms a statement.

An axiom is a proposition that is assumed to be true. Axioms form the basis on which we act to develop postulates. Axioms are not the immediate subject of investigation; however, the concepts used in axioms may also be used in postulates. They are challenged by measurements and can be shown to need redefinition. Then it becomes necessary to reassess what we initially considered as axioms. One difficulty in science is that very many propositions, both axioms and postulates, are classified as “true so far.”

If the propositions are as yet untested then they are postulates. Postulates are the fulcrum of the scientific method and can be investigated by direct investigation or experiment. Postulates demand investigation, but, once investigated, they may force the revision, or enable the extension, of established theory.

Data are the results of observations, surveys, or experiments made in relation to the investigation of specific postulates. Data do not exist in a logical vacuum and they are not neutral. The frequently heard expression, “Let the data speak for themselves,” can be badly misinterpreted. Data can only be used relative to the precise conditions of the investigation designed to address specific postulates.

A theory is a construction that evolves over time. It contains both axioms, which are generally thought to be true but may turn out not to be, and postulates, some of which are likely to be rejected as untrue. A theory contains questions.
The term hypothesis is reserved for use where a postulate can be examined in relation to data so that it can be designated true or false with some specified probability. In a statistical argument, the construction of hypotheses takes particular forms. A hypothesis can be solved for a logical answer. Not all postulates can be translated into hypotheses at the same time.

Whatever scientific theory is considered, there must be a set of questions which motivate it and a body of propositions, axioms together with postulates, that define it. As theories are considered on a broader level, specifying precise linkages between the motivating set of questions and defining body of propositions becomes more difficult. There are two linked reasons for this. The first is technical. It must always be possible to resolve the linkages between postulates and data by specific investigations. But general theories also have to include propositions that link sets of axioms built up from very specific investigations. These “linking propositions” can be difficult to test. They tend to require wide ranging investigations outside the scope of a single individual action, research experiment, or investigation. The second reason it is difficult to investigate broad theories is practical and sociological. Faced with the difficulty of connecting their research questions to the fundamentals of the theory, scientists tend to work within a general theory that really becomes a “viewpoint” or “paradigm.” Scientists, both as groups and individuals, have an over-arching axiom that specifies basic assumptions. When working within a paradigm, the logic behind the wider, more general, and linking propositions receives fewer direct challenges.

The first task when starting a scientific research project is to make a conceptual analysis. The scientific terms must be defined and the nature of their linkage in the formation of axioms and postulates must be specified. This involves a literature review and, from that, an analysis of the logical connections.

Consider scientific knowledge as a fish net that we construct, mend, and constantly use for different purposes. The concepts are the material that makes up the net, say the different types of filament, and the axioms are the way the filament is combined, including mesh size and the total arrangement of the net. The postulates are new or additional features of the net designed to catch a new category of fish.

Concepts, theorems, and postulates can be constructed at different levels of organization; for example, we use concepts to describe theories of subatomic physics, plate tectonics, and evolution. We must develop our own description of theory depending on the particular problem faced. The questions we ask determine the necessary set of theorems. Each question is different—there are no absolutes—but there is a common logical structure based upon the problematic situation and the need to define axioms and develop postulates.

Classification of Concepts

Axioms

Concepts by intellection are those that we use in our description of the more established parts of a theory. They have matured through use in argument and tests with real world data through either direct or indirect comparison with concepts by inspection. Concepts by intellection are repeatedly used in postulates in combination with concepts by imagination.

Appendix 3.1
Postulates

Concepts by imagination are those that we use in the development of postulates. They do not arise entirely through logical reasoning from within the current theory under examination. Concepts by imagination may originate by comparative reasoning. For example, a theory on the control of growth of species A may be “thought” likely to be sufficiently close to that of species B, a taxonomically related but more well-researched species, to form a model of it. However, an assumption is made about the similarity, that taxonomic criteria and growth control processes are closely related. Unless the two bodies of theory are tied together, the “borrowed” concept from species B is clearly a concept by imagination when used in a theory of the growth control of species A. Concepts by imagination are linked with concepts by intellection in postulates. They are compared with concepts by inspection, the result of data collection and analysis.

Data

Concepts by inspection are data used to examine whether a postulate is rejected. It is important not to assume that data have a one-to-one relationship with concepts by imagination or concepts by intellection. The technical process of measurement is frequently complicated. What is measured may only partly represent what the concept by imagination or intellection specified. Furthermore, comparisons may require a statistical framework. Observations and measurements have particular sampling and distributional properties that need to be clearly designated as a potential source of discrepancy between postulated and observed. So it is important to distinguish between concepts by imagination or intellection and those by inspection. The description of radiation by wavelengths is a concept by inspection.

General Observation

The involvement of non-specialist ideas in a theory is described by concepts by intuition. Northrop (1983) defines these as concepts that keep their meaning constant between scientists. The color blue is a concept by intuition. These concepts are widely accepted and may have no rigorously defined place in a particular theory under consideration.

The first requirement is to produce as precise a definition as possible of the problematic situation. We make a conceptual and propositional analysis of the theory. Of course the postulates will be of little value unless investigations can be implemented that will result in data and comparisons. This means analysis of what we think we know, axioms, and as rigorous a definition as possible of the postulates (figure 3.1.1). Although we may think of data as representing the real world, this representation is restricted by the postulates we construct, so it is essential to have them well-defined first.

We sometimes refer to data as facts. We can make the distinction between two types of facts, observed and described. It is important to note that there is no such thing as “pure fact.” The iterative nature of scientific investigation means that collecting data is always done with some proposition in mind, however basic it may be. If, for example, you choose to start an investigation by making a survey, the proposition exists that what you are surveying is important in the theory. Nothing is neutral. “Pure fact” cannot be communicated; to communicate you must describe it in a context. Ford would extend this argument further to say that there is no such thing as pure fact, and the term “immediately apprehended fact” is a better designator. It is a contradiction to say “Let the facts speak for themselves!” They do not exist other
than in association with a postulate. Measurements and observations should have information content, and it is the job of the scientist to maximize that information content. This can be done only if the limitations and explicit and implicit contexts of measurement and observation are understood.

Conceptual analysis highlights two important challenges in scientific research. First, we have to appreciate that our units of logical analysis, the concepts, are all of our own construction and, as such, they are very likely to change. The terms intellection, imagination, intuition, and inspection each describe mental processes, but ones that we ourselves may have difficulty in separating into precise activities. This is part of the second challenge of scientific research. It is a continuous process, where what we discover may not only deny our theories but almost inevitably will challenge the present construction. Our concepts by intuition and inspection, those we develop to describe the investigations, must be matched against concepts by imagination, those that we have developed to extend our theory. There is almost always a need for refinement of our concepts by imagination as we develop them into concepts by intellection, and this in turn demands adjustment in the meaning and relative importance of the concepts by intellection used in the construction of our original concepts by imagination.

Proof, Validation, and Verification

Some words used in scientific methodology need careful definition, and their use should be restricted to conditions under which that definition applies. These three words are sometimes used in casual conversation by scientists almost as interchangeable. This is regrettable. Unfortunately, we do not have a single word to describe the scientific method. These words have different meanings, and the differences are important for their application.

Definitions

Proof

That which establishes the truth of anything. Generally, proof is confined to the arguments of mathematical theorems. Proof is a formal, logical relationship between propositions. It is independent of the status of the concepts used in the construction of axioms and postulates.

Validation

Ratification, confirmation, substantiation. Clearly, science works by seeking confirmation in measurement of what has been considered in logical development. But that is NOT sufficient for the scientific method. Validation is something done to parking tickets—though that use of the word is also outside the dictionary definition. What is meant is that a merchant or business may certify that parking is free. In both cases, the attempt to use “validate” as a one-word description of a process violates the original meaning of the word. For parking tickets, one can (possibly) tolerate the violation as part of the charm of language development. In science, one cannot tolerate it.

Verification

Testifying, ascertaining, confirming, or testing the truth or accuracy of, asserting or proving to be true. While this definition of verification includes confirming,
confirmation is only part of the scientific method. The element of testing included in the definition of verification makes it wider than either validation or proof.

Ford gives a five-stage process of verification:

1. **Specification of primitive concepts by postulation.** These concepts designate the component entities and structures of the science.

2. **Derivation of compound postulates that specify logical relationships between concepts by intellection and by imagination.**

3. **Specification of systematic and unambiguous formal predictions.** It is at this stage that we may develop hypotheses, and we restrict the use of this work to an investigation or experiment that will have a conclusion with an ascribed probability based in statistical theory.

4. **Direct, controlled empirical experimentation produces data that are described with concepts by inspection.** A postulate is shown to be false when the predictions of stage 3 differ from what is observed in stage 4. The application of this five-stage process depends upon conducting controlled investigations or experimentation. We will consider cases where this is not possible and also where the conditions that are imposed upon an investigation to make it controlled, reduce its relevance to answering the original question.

5. **Formulation of rival propositions.** We can never prove that a proposition is true. In the logical statement

   \[
   \text{If } A, \text{ then } B
   \]

   our theory, represented by A, comprises propositions that use concepts by imagination and intellection. The scientific method is to examine rival propositions rigorously. This is the most difficult thing in science, but it is the only way in which we can advance the growth of objective knowledge. Sometimes rival propositions are explicit and exciting; sometimes they may require the diligent and routine investigation of a series of alternatives. An important problem is that we, as individuals, may become emotionally attached to the postulates we have proposed or axioms we like.

**Postulate Methods**

Ford gives an explanation of two postulate methods, the **multiple working postulates** and the **postulate through exploratory analysis**. The following is a summary of this explanation.

**Method of Multiple Working Postulates**

An essential element of the philosophy of scientific methodology is having multiple postulates. Because we cannot be sure of our propositions, we should examine different ones. The examination of multiple propositions can be considered to be a function of a community of scientists or a philosophy for the individual. Chamberlin (1965) advanced his philosophy for the individual in terms of the method of "multiple working hypotheses." He used the term "hypothesis" in its general and not restricted statistical sense. His article, written in 1890, compared the attitudes of working with multiple propositions to that of working with a single proposition or
that of working with a ruling theory. His plea was for scientists to keep their minds open. He considered that adopting a single proposition was akin to parenting and could lead to unconscious processes of selection and neglect of argument and debate in favor of the single proposition. Chamberlin suggested that multiple propositions are particularly valuable where complex explanations can be anticipated. He used as his example the geological processes that led to the formation of the Great Lakes. This could not be ascribed to one process—e.g., glaciation—but was probably the result of a number of contributing processes. This is an interesting example because in geology (as frequently in ecology) one is intent to develop a theory about a system where there may be multiple processes operating.

Exploratory Analysis
Postulates

There are three conditions under which we may consider exploratory analysis to be necessary:

(1) Inadequate definition of quantitative concepts.

We are not sure that the measurements (concepts by inspection) we intend to make will represent the concept by imagination effectively. Before we specify a hypothesis, we must be sure that our measurements will allow us to detect any effect of a treatment. Measurement accuracy requires a combination of accuracy of the instrument making the measurement and sufficient sampling to define the natural range of variation. It may be necessary to define the sampling distribution of measurements to see whether mathematical transformations will be required.

(2) Inadequate definition of relational concepts.

An important task in science is to define the relationship between concepts. When functional relationships are approximations, it may not be possible to resolve differences.

(3) Inadequate definition of the scope of the postulate.

In some cases, exploratory analysis may be necessary to investigate the scope of a postulate.

Loehle (1990) described this stage of the scientific method as theory maturation. This takes place during the development of postulates. In the early stages, it may be unclear what predictions can be made. Attempts to proceed too soon to develop a falsifiable or confirmatory hypothesis may lead to premature rejection of a partially correct theory. The objective of the maturation process should be to make postulates precise and predictive.

Conclusions

This review of the scientific method is an abridged version of Ford’s lectures. Specifically missing is the mathematics involved (e.g., testing the null hypothesis). This is beyond the scope of this guide, but this information is available in any undergraduate statistics text. In summary, the scientific method can be explained as a process whereby the scientist conceptually develops an idea; tests the idea through a battery of observations and measurements; rejects, accepts, or modifies the idea; and shows his or her proof for peer review (and hopefully acceptance).


APPENDIX 3.2
3.2 Procedures for Determining Unified Soil Classification

This appendix is divided in two parts.

- The first part, describing the visual method for unified soil classification, is extracted from *Visual Classification of Soils, Unified Soils Classification System* by A.K. Howard, 1986, U.S. Bureau of Reclamation Engineering and Research Center, Geotechnical Branch Training Manual No. 5.

- The second part, describing the laboratory method for unified soil classification, is adapted from *Laboratory Classification of Soils, Unified Soils Classification System* by A.K. Howard, 1986, U.S. Bureau of Reclamation Engineering and Research Center, Geotechnical Branch Training Manual No. 4.
Visual Method for Determining Unified Soil Classification
PROCEDURE FOR
DETERMINING UNIFIED SOIL CLASSIFICATION
(Visual Method)

INTRODUCTION
This procedure is under the jurisdiction of the Geotechnical Branch, code D-1340, Division of Research and Laboratory Services, EdR Center, Denver, Colorado. The procedure is issued under the fixed designation USBR 5005. The number immediately following the designation indicates the year of acceptance or the year of last revision.

This procedure is similar to ASTM D 2488, Standard Practice for Description and Identification of Soils (Visual- Manual Procedure) except for the following: (1) change in title and format, (2) references to USBR procedures, (3) note 2 in ASTM D 2488 is required in this procedure, (4) note 4 in ASTM D 2488 is required in this procedure, (5) the maximum particle size is in millimeters with prescribed increments of measurement, (6) the specimens for the dry strength test are one-fourth inch (6 mm) in diameter rather than one-half inch (12 mm), (7) note 14 is not used, and (8) moisture content is used here instead of water content and mass is substituted for weight.

For circumstances where it may be required or expedient to use ASTM standards, ASTM D 2487 or D 2488 may be substituted for USBR 5000 or 5005, respectively. However, it must be clearly stated in written comments, tables, figures, and logs that the ASTM standards were used.

1. Scope

1.1 This designation outlines the procedures for the description of soils for engineering purposes.
1.2 This designation outlines procedures for visually identifying soils for engineering purposes based on the classification system described in USBR 5000. The identification is based on visual examination and manual tests. It must be clearly stated in reporting an identification that it is based on the visual-manual process.
1.2.1 When precise classification of soils for engineering purposes is required, the procedures required in USBR 5000 shall be used.
1.2.2 The identification portion of this procedure — in assigning a group symbol and name — is limited to soil particles smaller than 5 inches (75 mm); that is, passing a U.S.A. Standard series 3-inch sieve.
1.2.3 The identification portion of this procedure is limited to naturally occurring soils.

NOTE 1.—This procedure may be used as a descriptive system applied to such materials as shale, claystone, shales, crushed rock, etc. (see app. X2).

1.3 The descriptive information in this procedure may be used with other soil classification systems or for materials other than naturally occurring soils.

2. Applicable Documents

2.1 USBR Procedures:
USBR 5000 Determining Unified Soil Classification
(Experimental Method)
2.2 ASTM Standards:
D 2487 Classification of Soils for Engineering Purposes
D 2488 Standard Practice for Description and Identification of Soils (Visual- Manual Procedure)

3. Summary of Method

3.1 Using visual examination and simple manual tests, this procedure gives standardized criteria and processes for describing and identifying soils.
3.2 Soil can be given an identification by assigning a group symbol(s) and name. The flow charts (figure 1 for fine-grained soils and figure 2 for coarse-grained soils) can be used to determine the appropriate group symbol(s) and name. If the soil has visually determined properties that do not distinctly place it into a specific group, borderline symbols may be used (see app. X3).
3.3 A distinction must be made between dual symbols and borderline symbols.
3.3.1 A dual symbol (two symbols separated by a hyphen, e.g., GP-GM, SW-SC, CL-ML) should be used to indicate the soil has been identified as having the properties of a classification as required by USBR 5000 where two symbols are required. Two symbols are required when the soil has between 5 and 12 percent fines and where the liquid limit and plasticity index values plot in the CL-ML (cross hatched) area of the plasticity chart.
3.3.2 A borderline symbol (two symbols separated by a slash, e.g., CL/CH, GM/SM, CL/ML) should
Figure 1a. - Flowchart for identifying inorganic fine-organic fine-grained soil (50% or more fines) — visual-manual method.

Figure 1b. - Flowchart for identifying organic fine-grained soil (50% or more fines) — visual-manual method
Figure 2 - Flowchart for identifying coarse-grained soil (less than 50% fines) — visual-manual method.
be used to indicate the soil has been identified as having properties that do not distinctly place the soil into a specific group (see app. X3).

4. Significance and Use

4.1 The descriptive information required in this procedure can be used to describe a soil to aid in the evaluation of its significant properties for engineering use.

4.2 The descriptive information required in this procedure should be used to supplement the classification of a soil as determined in USBR 5000.

4.3 This procedure may be used in identifying soils using the classification group symbols and names as prescribed in USBR 5000. Since the names and symbols used in this procedure to identify the soils are the same as those used in USBR 5000, it shall be clearly stated in reports, etc., that the classification symbol and name are based on the usual manual procedures.

4.4 This procedure is to be used not only for identification of soils in the field but also in the office, in the laboratory, or wherever soil samples are inspected and described.

4.5 The procedure has particular value in grouping similar soil samples so that only a minimum number of laboratory tests need be run for positive soil classification.

NOTE 2.—The ability to describe and identify soils correctly is learned more readily under the guidance of experienced personnel, but it may also be acquired systematically by comparing numerical laboratory test results for typical soils of each type with their usual and manual characteristics.

4.6 When describing and identifying soil samples from a given bore, test pit, or group of borings or pits, it is not necessary to follow all of the processes in this procedure for every sample. Soils which appear to be similar can be grouped together. One sample from the group can be completely described and identified, with the others referred to "as similar" based on performing only a few of the descriptive and identification processes described in this procedure.

5. Terminology

5.1 Definitions are in accordance with USBR 3900.

Terms of particular significance are:

5.1.1 Boulder.—A particle of rock that will not pass a 12-inch (300-mm) square opening.

5.1.2 Cobble.—A particle of rock that will pass a 12-inch (300-mm) square opening and be retained on a U.S.A. Standard 3-inch (75-mm) sieve.

5.1.3 Peat.—A soil primarily composed of vegetable tissue in various stages of decomposition with an organic odor, a dark brown to black color, a spongy consistency, and a texture ranging from fibrous to amorphous (ASTM D 2487-83).

5.2 Terms Specific to This Designation:

5.2.1 Gravel.—Particles of rock that will pass a 3-inch (75-mm) sieve and be retained on a No. 4 (4.75-mm) sieve with the following subdivisions:

<table>
<thead>
<tr>
<th>Coarse</th>
<th>Passes 3-inch (75-mm) sieve and retained on 3/4-inch (19.0-mm) sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine</td>
<td>Passes 3/4-inch (19.0-mm) sieve and retained on No. 4 (4.75-mm) sieve</td>
</tr>
</tbody>
</table>

5.2.2 Sand.—Particles of rock that will pass a No. 4 (4.75-mm) sieve and be retained on a No. 200 (75-μm) sieve with the following subdivisions:

<table>
<thead>
<tr>
<th>Coarse</th>
<th>Passes No. 4 (4.75-mm) sieve and retained on No. 10 (2.00-mm) sieve</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium</td>
<td>Passes No. 10 (2.00-mm) sieve and retained on No. 40 (425-μm) sieve</td>
</tr>
<tr>
<td>Fine</td>
<td>Passes No. 40 (425-μm) sieve and retained on No. 200 (75-μm) sieve</td>
</tr>
</tbody>
</table>

5.2.3 Clay.—Soil passing the No. 200 (75-μm) U.S.A. Standard sieve that exhibits plasticity (potty-like properties) within a range of moisture contents, and which exhibits considerable strength when air-dry. For classification, a clay is a fine-grained soil, or the fine-grained portion of a soil, having a plasticity index equal to or greater than 4 and the plot of plasticity index versus liquid limit falls on or above the "A"-line (see fig. 3, USBR 5000).

5.2.4 Silt.—Material passing the No. 200 (75-μm); U.S.A. Standard sieve that is nonplastic or very slightly plastic and that exhibits little or no strength when air-dry (ASTM). For classification, a silt is a fine-grained soil, or the fine-grained portion of a soil, having a plasticity index less than 4 or if the plot of plasticity index versus liquid limit falls below the "A"-line (see fig. 3, USBR 5000).

5.2.5 Organic Clay.—A clay with sufficient organic content to influence the soil properties. For classification, an organic clay is a soil that would be classified as a clay except that its liquid limit value after overdrying is less than 75 percent of its liquid limit value before overdrying.

5.2.6 Organic Silt.—A silt with sufficient organic content to influence the soil properties. For classification, an organic silt is a soil that would be classified as a silt except that its liquid limit value after overdrying is less than 75 percent of its liquid limit value before overdrying.

6. Apparatus

6.1 Required Apparatus:

6.1.1 Small supply of water.
6.1.2 Pocket knife or small spatula.

6.2 Useful Auxiliary Apparatus:

6.2.1 Small bottle of dilute hydrochloric acid, one part HCl (10 N) to three parts distilled water.
6.2.2 Small test tube and stopper, or jar with a lid.
6.2.3 Dish for wash test.
6.2.4 Small hand lens.
6.2.5 Ruler.
7.1 When preparing the dilute HCl (hydrochloric acid) solution of one part concentrated HCl (10 N) to three parts of distilled water, slowly add acid into water following necessary safety precautions. Handle with caution and store safely. If solution comes in contact with skin, rinse thoroughly with water.

**CAUTION:** Do not add water to acid.

8. Sampling

8.1 The sample shall be considered to be representative of the stratum, from where it was obtained, by an appropriate accepted or standard procedure.

**NOTE 3:** The sampling procedure should be identified as having been conducted in accordance with a USBR procedure or an ASTM standard, or other appropriate standard or procedure.

8.2 The sample shall be carefully identified as to origin.

**NOTE 4:** Remarks as to the origin may take the form of a boring number and sample number in conjunction with a job number, a geologic stratum, a geologic horizon, or a location description with respect to a permanent monument, grid system, or section number and offset with respect to a stated coordinate, and a depth or elevation.

8.3 For accurate description and identification, the minimum amounts of the specimen to be examined shall be in accordance with the following schedule:

<p>| Maximum particle size, | Minimum specimen size, |
| sieve opening | dry mass |</p>
<table>
<thead>
<tr>
<th>mm</th>
<th>kg</th>
<th>lbm</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.75</td>
<td>0.1</td>
<td>0.2</td>
</tr>
<tr>
<td>9.5</td>
<td>0.25</td>
<td>0.5</td>
</tr>
<tr>
<td>19.0</td>
<td>1.1</td>
<td>2.5</td>
</tr>
<tr>
<td>37.5</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>75.0</td>
<td>70</td>
<td>150</td>
</tr>
</tbody>
</table>

**NOTE 5:** If random, isolated particles are encountered that are significantly larger than the particles in the soil matrix, the soil matrix can be accurately described and identified in accordance with the above schedule.

8.4 If the field sample or specimen being examined is smaller than the minimum amount, the report shall include an appropriate remark.

9. Descriptive Information

9.1 Describe the **angularity** of the sand (coarse sizes only), gravel, cobbles, and boulders as rounded, subrounded, subangular, or angular as indicated by the criteria in table 1 and on figure 5. A range of angularity may exist such as subrounded to rounded.

<table>
<thead>
<tr>
<th>Angularity</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rounded</td>
<td>Particles have smoothly curved sides and no edges</td>
</tr>
<tr>
<td>Subrounded</td>
<td>Particles have nearly plane sides but have well-rounded corners and edges</td>
</tr>
<tr>
<td>Subangular</td>
<td>Particles are similar to angular description but have rounded edges</td>
</tr>
<tr>
<td>Angular</td>
<td>Particles have sharp edges and relatively plane sides with unpolished surfaces</td>
</tr>
</tbody>
</table>

**Figure 5:** Typical angularity of coarse-grained particles.

9.2 Describe the **shape** of the gravel, cobbles, and boulders as flat, elongated, or flat and elongated if they meet the criteria in table 2 and on figure 4; otherwise, do not remark. Indicate the fraction of particles having that shape such as one-third of gravel particles are flat.

<table>
<thead>
<tr>
<th>Shape</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flat</td>
<td>Particles with length &gt; 3 ( \frac{\text{width}}{\text{thickness}} )</td>
</tr>
<tr>
<td>Elongated</td>
<td>Particles with ( \frac{\text{length}}{\text{width}} ) &gt; 3</td>
</tr>
<tr>
<td>Flat and elongated</td>
<td>Particles meet criteria for both flat and elongated</td>
</tr>
</tbody>
</table>

**Figure 4:** Typical shape of coarse-grained particles.

**Table 2:** Criteria for describing particle shape (see fig. 4).

The particle shape shall be described as follows where length, width, and thickness refer to the greatest, intermediate, and least dimensions of a particle, respectively.

---

Appendix 3.2
For intact fine-grained soil, describe the consistency as very soft, soft, firm, hard, or very hard as indicated by the criteria in Table 3. This observation is inappropriate for disturbed soils or soils with significant amounts of gravel.

<table>
<thead>
<tr>
<th>Criteria for describing consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft</td>
</tr>
<tr>
<td>Soft</td>
</tr>
<tr>
<td>Firm</td>
</tr>
<tr>
<td>Hard</td>
</tr>
<tr>
<td>Very hard</td>
</tr>
</tbody>
</table>

Describe the cementsation of intact coarse-grained soils as weak, moderate, or strong as indicated by the criteria in Table 6.

<table>
<thead>
<tr>
<th>Criteria for describing cementsation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weak</td>
</tr>
<tr>
<td>Moderate</td>
</tr>
<tr>
<td>Strong</td>
</tr>
</tbody>
</table>

Describe the structure of intact soils according to the criteria in Table 7.

<table>
<thead>
<tr>
<th>Criteria for describing structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stratified</td>
</tr>
<tr>
<td>Laminated</td>
</tr>
<tr>
<td>Slikensided</td>
</tr>
<tr>
<td>Blocky</td>
</tr>
<tr>
<td>Leached</td>
</tr>
<tr>
<td>Homogeneous</td>
</tr>
</tbody>
</table>

For gravel and sand components, describe the ranges of particle sizes within each component as defined in subparagraphs 5.2.1 and 5.2.2 (for example: about 20 percent fine to coarse gravel, about 40 percent fine to coarse sand).

Describe the maximum particle size found in the sample.

If the maximum particle size is a sand size, describe as fine, medium, or coarse as defined in subparagraph 5.2.2 (for example: maximum size, medium sand).

If the maximum particle size is a gravel size.
10. Identification of Peat

10.1 A sample composed primarily of vegetable tissue in various stages of decomposition that has a fibrous to amorphous texture—usually a dark brown to black color—and an organic odor should be designated as a highly organic soil and shall be identified as peat, PT, and not subjected to the identification procedures described hereafter.

11. Specimen Preparation for Identification

11.1 The soil identification portion of this procedure is based on the minus 3-inch (75-mm) particle sizes. The plus 3-inch (75-mm) particles must be removed manually, for a loose sample, or mentally evaluated, for an intact sample before classifying the soil.

11.2 Estimate and note the percentage of cobbles and the percentage of boulders. Performed visually, these estimates will be on the basis of volume percentage.

NOTE 6.—Since the percentages of the particle-size distribution in USBR 3000 are by dry mass and the estimates of percentages for gravel, sand, and fines in this procedure are by dry mass, it is recommended that the report state that the percentages of cobbles and boulders are by volume.

11.3 Of the fraction of the soil smaller than 3 inches (75 mm), estimate and note the percentage, by dry mass, of the gravel, sand, and fines. (See app. X4 for suggested procedures.)

NOTE 7.—Since the particle-size components appear visually on the basis of volume, considerable experience is required to estimate the percentages on the basis of dry mass. Frequent comparisons with laboratory gradation analyses should be made.

11.3.1 The percentages shall be estimated to the nearest 5 percent. The percentages of gravel, sand, and fines must add up to 100 percent.

11.3.2 If one of the components is present, but not in sufficient quantity to be considered 5 percent of the minus 3-inch (75-mm) portion, indicate its presence by the term trace (for example: trace of fines). A trace is not to be considered in the total of 100 percent for the components.

12. Preliminary Identification Procedure

12.1 The soil is fine grained if it contains 50 percent or more fines; follow paragraph 13.

12.2 The soil is coarse grained if it contains less than 50 percent fines; follow paragraph 14.

13. Procedure for Identifying Fine-Grained Soils

13.1 Selection.—Select a representative sample of the material for examination. Remove particles larger than the No. 40 sieve (medium sand and larger) until a specimen equivalent to about a handful of material is available. Use this specimen for performing the dry strength, dilatancy, and toughness tests.

13.2 Dry Strength.—From the specimen, select enough material to mold into a ball about 1 inch (25 mm) in diameter. Mold the material until it has the consistency of putty; add water if necessary.

13.2.1 From the molded material, make at least three test specimens. A test specimen shall be a ball of material about 1/4 inch (6 mm) in diameter. Allow the test specimens to dry in air or sun or dry by artificial means as long as the temperature does not exceed 140 °F (60 °C).

13.2.2 If the test specimen contains natural dry lumps, those that are about 1/4 inch (6 mm) in diameter may be used in place of the molded balls.

NOTE 8.—The process of molding and drying usually produces higher strengths than are found in natural dry lumps of soil.

13.2.3 Test the strength of the dry balls or lumps by crushing between the fingers and note the strength as none, low, medium, high, or very high according to the criteria in table 8. If natural dry lumps are used, do not use the results of any of the lumps that are found to contain particles of coarse sand.

<table>
<thead>
<tr>
<th>Table 8. - Criteria for describing dry strength.</th>
</tr>
</thead>
<tbody>
<tr>
<td>None</td>
</tr>
<tr>
<td>Low</td>
</tr>
<tr>
<td>Medium</td>
</tr>
<tr>
<td>High</td>
</tr>
<tr>
<td>Very High</td>
</tr>
</tbody>
</table>
13.2.41 The presence of high-strength water-soluble cementing materials, such as calcium carbonate, may cause exceptionally high dry strengths. The presence of calcium carbonate usually can be detected from the intensity of the reaction with dilute hydrochloric acid (see subpar. 9.6).

13.3 Dilacancy.—From the specimen, select enough material to mold into a ball about 1/2 inch (12 mm) in diameter. Mold the material, add water if necessary until it has a soft, but not sticky, consistency.

13.3.1 Smooth the soil ball in the palm of one hand with the blade of a knife or small spatula. Shake horizontally, striking the side of the hand vigorously against the other hand several times. Note the reaction of water appearing on the surface of the soil. Squeeze the sample by closing the hand or pinching the soil between the fingers and note reaction as none, slow, or rapid according to the criteria in Table 9. The reaction is the speed at which water appears while shaking and disappears while squeezing.

<table>
<thead>
<tr>
<th>Table 9 — Criteria for describing dilacancy.</th>
</tr>
</thead>
<tbody>
<tr>
<td>None — No visible change in the specimen</td>
</tr>
<tr>
<td>Slow — Water appears slowly on the surface of the specimen during shaking and does not disappear or disappears slowly upon squeezing</td>
</tr>
<tr>
<td>Rapid — Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing</td>
</tr>
</tbody>
</table>

13.4 Toughness.—Following completion of the dilacancy test, shape the test specimen into an elongated pat and roll by hand on a smooth surface or between the palms into a thread about 1/8 inch (3 mm) in diameter. (If the sample is too wet to roll easily, it should be spread out into a thin layer and allowed to lose some water by evaporation.) Fold the sample threads and roll repeatedly until the thread crumbles to a diameter of about 1/8 inch (3 mm). The thread will crumble at a diameter of 1/8 inch (3 mm) when the soil is near the plastic limit. Note the pressure required to roll the thread near the plastic limit. Also, note the strength of the thread. After the thread crumbles, the pieces should be lumped together and kneaded until the lump crumbles. Note the toughness of the material during kneading.

13.4.1 Describe the toughness of the thread and lump as low, medium, or high according to the criteria in Table 10.

<table>
<thead>
<tr>
<th>Table 10 — Criteria for describing toughness.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low — Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft.</td>
</tr>
<tr>
<td>Medium — Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium toughness.</td>
</tr>
<tr>
<td>High — Considerable pressure is required to roll the thread to near the plastic limit. The thread and the lump have very high toughness.</td>
</tr>
</tbody>
</table>

13.5 Plasticity.—On the basis of observations made during the toughness test, describe the plasticity of the material according to the criteria given in Table 11.

<table>
<thead>
<tr>
<th>Table 11 — Criteria for describing plasticity.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nonplastic — A 1/8-inch (3-mm) thread cannot be rolled at any moisture content.</td>
</tr>
<tr>
<td>Low — The thread can be barely rolled and the lump cannot be formed when drier than the plastic limit.</td>
</tr>
<tr>
<td>Medium — The thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.</td>
</tr>
<tr>
<td>High — It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times close to the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.</td>
</tr>
</tbody>
</table>

13.6 Inorganic/Organic.—Decide whether the soil is an inorganic or an organic fine-grained soil (see subpar. 13.8). If inorganic, follow subparagragh 13.7.

13.7 Identification of Inorganic Fine-Grained Soils:

13.7.1 Identify the soil as a lean clay, CL, if the soil has medium to high dry strength, none to slow dilacancy, and medium toughness and plasticity (see Table 12).

13.7.2 Identify the soil as a fat clay, CH, if the soil has high to very high dry strength, no dilacancy, and high toughness and plasticity (see Table 12).

13.7.3 Identify the soil as a silt, ML, if the soil has none to low dry strength, slow to rapid dilacancy, and low toughness and plasticity or is nonplastic (see Table 12).

13.7.4 Identify the soil as an elastic silt, MH, if the soil has low to medium dry strength, none to slow dilacancy, and low to medium toughness and plasticity (see Table 12).

NOTE 9.—These properties for elastic silt are similar to those for a lean clay. However, the silt will dry much faster on the hand and have a smooth, silky feel when dry. Some soils which would classify as elastic silt, MH, according to the criteria in USBR 5000 are visually difficult to distinguish from lean clay, CL. It may be necessary to perform laboratory testing for proper identification.

Table 12 — Identification of inorganic fine-grained soils from manual tests.

<table>
<thead>
<tr>
<th>Soil symbol</th>
<th>Dry strength</th>
<th>Dilacancy</th>
<th>Toughness</th>
</tr>
</thead>
<tbody>
<tr>
<td>ML</td>
<td>None to low</td>
<td>Slow to rapid</td>
<td>Low or thread cannot be formed</td>
</tr>
<tr>
<td>CH</td>
<td>High to very high</td>
<td>None</td>
<td>Medium</td>
</tr>
<tr>
<td>CL</td>
<td>Medium to high</td>
<td>None to slow</td>
<td>Low to medium</td>
</tr>
<tr>
<td>MH</td>
<td>Low to medium</td>
<td>None to slow</td>
<td>High</td>
</tr>
</tbody>
</table>
13.8 Identification of Organic Fine-Grained Soils:

13.8.1 Identify the soil as an organic soil, OL or OH, if the soil contains enough organic particles to influence the soil properties. Organic soils usually have a dark brown to black color and may have an organic odor. Often, organic soils will change color, e.g., black to brown, when exposed to the air. Some organic soils will lighten in color significantly when air dried. Normally, organic soils would not have a high toughness or plasticity. The thread for the toughness test will be spongy.

NOTE 10—In some cases, through practice and experience, it may be possible to further identify the organic soils as organic silts or organic clays, OL or OH. Correlations between the dilatancy, dry strength, and toughness tests and laboratory tests can be made to identify organic soils in certain deposits of similar materials of known geologic origin.

13.9 If the soil is estimated to have 15 to 25 percent sand or gravel or both, the words "with sand" or "with gravel" shall be added to the group name (see figs. 1a and 1b) (for example: lean clay with sand, CL; silt with gravel, ML). If the percent of sand is equal to the percent of gravel, use "with sand." If the soil is estimated to have 30 percent or more sand or gravel or both, the words "sandy" or "gravely" shall be added to the group name. Add the word "sandy" if there appears to be more sand than gravel. Add the word "gravely" if there appears to be more gravel than sand (see figs. 1a and 1b) (for example: sandy lean clay, CL; gravelly fine clay, CH; sandy silt, ML). If the percent of sand is equal to the percent of gravel, use "sandy.

14. Procedure for Identifying Coarse-Grained Soils (contains less than 50 % fines)

14.1 The soil is a gravel if the percent gravel is estimated to be more than the percent sand.

14.2 The soil is a sand if the percent gravel is estimated to be equal to or less than the percent sand.

14.3 The soil is a clean gravel or clean sand if the percent fines is estimated to be 5 percent or less.

14.3.1 Identify the soil as a well-graded gravel, GW, or as a well-graded sand, SW, if it has a wide range of particle sizes and substantial amounts of the intermediate particle sizes.

14.3.2 Identify the soil as a poorly graded gravel, GP, or as a poorly graded sand, SP, if it consists predominantly of one size (uniformly graded) or if it has a wide range of sizes with some intermediate sizes obviously missing (gap or skip graded).

14.4 The soil is either a gravel with fines or a sand with fines if the percent fines is estimated to be 15 percent or more.

14.4.1 Identify the soil as a clayey gravel, GC, or a clayey sand, SC, if the fines are clayey as determined by the procedures in paragraph 13.

14.4.2 Identify the soil as a silty gravel, GM, or a silty sand, SM, if the fines are silty as determined by the procedures in paragraph 13.

14.5 If the soil is estimated to contain 10 percent fines, give the soil a dual identification using two group symbols.

14.5.1 The first group symbol shall correspond to a clean gravel or sand (GW, GP, SW, SP) and the second symbol shall correspond to a gravel or sand with fines (GC, GM, SC, SM).

14.5.2 The group name shall correspond to the first group symbol plus "with clay" or "with silt" to indicate the plasticity characteristics of the fines (see fig. 3) (for example: well-graded gravel with clay, GW-SC, poorly graded sand with silt, SP-SM).

14.6 If the specimen is predominantly sand or gravel but contains an estimated 15 percent or more of the other coarse-grained constituent, the words "with gravel" or "with sand" shall be added to the group name (see fig. 3) (for example: poorly graded gravel with sand, GP: clayey sand with gravel, SC).

14.7 If the field sample contained any cobbles and/or boulders, the words "with cobbles," or "with cobbles and boulders" shall be added to the group name (for example: silty gravel with cobbles, GM).

15. Report

15.1 The report shall include information as to sample origin as well as the items indicated in table 13.

NOTE 11—Example: CLAYEY GRAVEL WITH SAND AND COBBLES (GC): About 30 percent fine to coarse, sub-rounded to subangular gravel; about 30 percent fine to coarse, sub-rounded sand; about 20 percent fines with medium plasticity, high dry strength, no dilatancy, medium toughness; original field sample had trace of hard, subrounded cobbles: maximum size, 150 mm; weak reaction with HCl.

In-place conditions: firm, homogeneous, dry, brown
Geologic interpretation: alluvial fan

NOTE 12.—Other examples of soil descriptions and identifications are given in appendices X1 and X2.

15.2 If, in the soil description, the soil is identified using a classification group symbol and name as described in USBR 5000, it must be distinctly and clearly stated in log forms, summary tables, reports, etc., that the symbol and name are based on visual-manual procedures.

16. Precision and Accuracy

16.1 This method provides qualitative information only; therefore, a precision and accuracy statement is nonapplicable.
Table 13 - Checklist for description of soils

1. Group name
2. Group symbol
3. Percent of cobbles and/or boulders (by volume)
4. Percent of gravel, sand and/or fines (by dry mass)
5. Particle size range: Gravel - fine, coarse
   Sand - fine, medium, coarse
6. Particle angularity: angular subangular subrounded rounded
7. Particle shape (if appropriate): flat elongated flat and elongated
8. Maximum particle size or dimension
9. Hardness of coarse sand and larger particles
10. Plastics of fines: nonplastic low medium high
11. Dry strength: none low medium high very high
12. Dilatancy: none slow rapid
13. Toughness: low medium high
14. Color (in moist condition)
15. Odor — mention only if organic or unusual
16. Moisture: dry moist wet
17. Reaction with HCl: none weak strong

For intact samples:
18. Consistency (fine-grained soils only): very soft soft firm hard very hard
19. Structure: stratified laminated fissured slickensided lensed homogeneous
20. Cementation: weak moderate strong
21. Local name
22. Geologic interpretation

Additional comments:
- Presence of roots or root holes
- Presence of nips, pyramids, etc.
- Surface coatings on coarse-grained particles
- Caving or sloughing of auger hole or trench sides
- Difficulty in augering or excavation
- Etc.
APPENDIX

X1. EXAMPLES OF VISUAL SOIL DESCRIPTIONS

X1.1 The following examples show how the information required in subparagraph 15.1 can be reported. The information that is included in descriptions should be based on individual circumstances and need.

Example 1: WELL-GRADED GRAVEL WITH SAND (GW): About 75 percent fine to coarse, hard, subangular gravel; about 25 percent fine to coarse, hard, subangular sand; trace of fines; maximum size, 75 mm; dry, brown; no reaction with HCl.

Example 2: SILTY SAND WITH GRAVEL (SM): About 60 percent predominantly fine sand; about 25 percent fines with low plasticity, low dry strength, rapid dilatancy, low toughness; about 15 percent fine, hard, subrounded gravel (a few gravel-size particles fractured by hammer blow); maximum size, 20 mm; no reaction with HCl. Note: field sample size smaller than recommended.

In-place conditions — firm, stratified and consists lenses of silt 1 to 2 inches thick, moist, brown to gray; in-place dry unit weight was 106 lb/ft³ and in-place moisture was 9 percent.

Example 3: ORGANIC SOIL (OL/OD): About 100 percent fines with low plasticity, slow dilatancy, low dry strength, low toughness; wet, dark brown, organic odor; weak reaction with HCl.

Example 4: SILTY SAND WITH ORGANIC FINES (SM): About 75 percent fine to coarse, hard, subangular reddish sand; about 25 percent organic and dark brown nonplastic fines; no dry strength, slow dilatancy; wet, maximum size, coarse sand; weak reaction with HCl.

Example 5: POORLY GRADED GRAVEL WITH SILT, SAND, COBBLES AND BOULDERS (GP-GM): About 75 percent fine to coarse, hard, subrounded to subangular gravel; about 15 percent fine, hard, subrounded to subangular sand; about 10 percent nonplastic fines; moist, brown; no reaction with HCl. Original field sample had a trace of hard, subrounded cobbles and a trace of hard, subrounded boulders, having a maximum dimension of 500 mm.

X2. USING THE IDENTIFICATION METHOD AS A DESCRIPTIVE SYSTEM FOR SHALE, CLAYSTONE, SHELLS, SLAG, CRUSHED ROCK, ETC.

X2.1 The identification method may be used as a descriptive system applied to materials that exist in situ as shale, claystone, sandstone, siltstone, mudstone, etc., but convert to soils after field or laboratory processing (crushing, slaking, etc.).

X2.2 Materials such as shells, crushed rock, slag, etc., should be identified as such. However, the processes used in this procedure for describing the particle size and plasticity characteristics may be used in the description of the material. If desired, an identification using a group name and symbol according to this method may be assigned to aid in describing the material.

X2.3 The group symbol(s) and group names should be placed in quotation marks or noted with some type of distinguishing symbol (see examples).

X2.4 Examples of how group names and symbols could be incorporated into a descriptive system for materials that are not naturally occurring soils follow.

Example 1: SHALE CHUNKS: Retrieved as 2- to 4-inch pieces of shale from power auger hole, dry, brown, no reaction with HCl. After slaking in water for 24 hours, material identified as "SANDY LEAN CLAY (CL)" — About 60 percent fines with medium plasticity, high dry strength, no dilatancy, medium toughness; about 35 percent fine to medium sand; about 5 percent gravel-size pieces of shale.

Example 2: CRUSHED SANDSTONE: Product of commercial crushing operation; "POORLY GRADED SAND WITH SILT (SP-SM)" — About 90 percent fine to medium sand; about 10 percent nonplastic fines; maximum size, medium sand; dry, reddish-brown; strong reaction with HCl.

Example 3: BROKEN SHELLS: Natural deposit of shells; "POORLY GRADED GRAVEL WITH SAND (GP)" — About 60 percent gravel-size broken shells; about 35 percent sand and sand-size shell pieces; about 5 percent fines.

Example 4: CRUSHED ROCK: Processed from gravel and cobbles in Pit No. 7; "POORLY GRADED GRAVEL (GP)" — About 90 percent fine, hard,angular gravel-size particles; about 10 percent coarse, hard, angular sand-size particles; maximum size, 20 mm; dry, tan; no reaction with HCl.
X3. SUGGESTED PROCEDURE FOR USING A BORDERLINE SYMBOL FOR SOILS WITH TWO POSSIBLE IDENTIFICATIONS

X3.1 Since this practice is based on estimates of particle size distribution and plasticity characteristics, it may be difficult to clearly identify the soil as belonging to one category. To indicate that the soil may fall into one of two possible basic groups, a borderline symbol may be used with the two symbols separated by a slash (for example: SC/CL, CL/CH).

X3.1.1 A borderline symbol may be used when the percent fines is estimated to be between 45 and 55 percent. One symbol should be for a coarse-grained soil with fines and the other for a fine-grained soil (for example: GM/ML, CL/SC).

X3.1.2 A borderline symbol may be used when the percent sand and the percent gravel is estimated to be about the same (for example: GP/SP, SC/GC, GM/SM). It is practically impossible to have a soil that would have a borderline symbol of GW/SW.

X3.1.3 A borderline symbol may be used when the soil could be either well graded or poorly graded (for example: GW/GP, SW/SP).

X3.1.4 A borderline symbol may be used when the soil could either be a silt or a clay (for example: CL/ML, CH/MH, SC/SM).

X3.1.5 A borderline symbol may be used when a fine-grained soil has properties that indicate that it is at the boundary between a soil of low compressibility and a soil of high compressibility (for example: CL/CH, MH/ML).

X3.2 The order of the borderline symbols should reflect similarity to surrounding or adjacent soils (for example: soils in a borrow area have been identified as CH. One sample is considered to have a borderline symbol of CL and CH. To show similarity, the borderline symbol should be CH/CL).

X3.3 The group name for a soil with a borderline symbol should be the group name for the first symbol except for:

- CL/CH - lean to fat clay
- ML/CL - clayey silt
- CL/ML - silty clay

X3.4 The use of a borderline symbol should not be used indiscriminately. Every effort should be made to place the soil into a single group.

X4. SUGGESTED PROCEDURES FOR ESTIMATING THE PERCENT OF GRAVEL, SAND, AND FINES IN A SOIL SAMPLE

X4.1 Jar Method.—The relative percentage of coarse- and fine-grained material may be estimated by thoroughly shaking a mixture of soil and water in a test tube or jar, and then allowing the mixture to settle. The coarse particles will fall to the bottom and successively finer particles will be deposited with increasing time; the sand sizes will fall out of suspension in 20 to 30 seconds. The relative proportions can be estimated from the relative volume of each size separate. This method should be correlated to particle-size laboratory determinations.

X4.2 Visual Method.—Mentally visualize the gravel size particles placed in a sack (or other container) or sacks. Then, do the same with the sand size particles and the fines. Then, mentally compare the number of sacks to estimate the percentage of plus No. 4 sieve size and minus No. 4 sieve size present. The percentages of sand and fines in the minus sieve size No. 4 material then can be estimated from the wash test (see subpar. X4.3).

X4.3 Wash Test (for relative percentages of sand and fines).—Select and moisten enough minus No. 4 sieve size material to form a 1-inch (25-mm) cube of soil. Cut the cube in half, set one-half to the side, and place the other half in a small dish. Wash and decant the fines out of the material in the dish until the wash water is clear, and then compare the two samples and estimate the percentage of sand and fines. Remember that the percentage is based on mass, not volume. However, the volume comparison will provide a reasonable indication of grain size percentages.

While washing, it may be necessary to break down lumps of fines with a finger to get the correct percentages.
Laboratory Method for Determining Unified Soil Classification

Prior to classifying a soil visually, it is necessary to understand the laboratory procedure for soil classification.

The basic purpose of the laboratory or the visual classification method is to place a soil into one of the basic groups, or into a borderline case between two of the basic groups, and to give it the appropriate group symbol and group name. The group symbols and names in the Unified Soil Classification System (USCS) serve as a convenient “shorthand” description of a soil. However, there are many other characteristics of soils that are important and must be communicated. A visual classification of a soil includes not only a group symbol and name, but a complete word description. This description is necessary because the person using the information generally does not have laboratory test data available.

Thus, the soil classifier must use a word picture to describe the soil, generally on a log form. A log is a written description of the soil obtained from drill holes, auger holes, test pits, or other excavations referenced to the depth interval from which the soil was obtained. A typical log is shown as figure 3.2.1. These classifications are generally performed using the visual method.

<table>
<thead>
<tr>
<th>GW</th>
<th>Well-graded gravel</th>
<th>ML</th>
<th>Silt</th>
</tr>
</thead>
<tbody>
<tr>
<td>GP</td>
<td>Poorly graded gravel</td>
<td>CL</td>
<td>Lean clay</td>
</tr>
<tr>
<td>GM</td>
<td>Silty gravel</td>
<td>MH</td>
<td>Elastic silt</td>
</tr>
<tr>
<td>GC</td>
<td>Clayey gravel</td>
<td>CH</td>
<td>Fat clay</td>
</tr>
<tr>
<td>SW</td>
<td>Well-graded sand</td>
<td>OL</td>
<td>Organic silt or organic clay</td>
</tr>
<tr>
<td>SP</td>
<td>Poorly graded sand</td>
<td>OH</td>
<td>Organic silt or organic clay</td>
</tr>
<tr>
<td>SM</td>
<td>Silty sand</td>
<td>PT</td>
<td>Peat and other highly organic soils</td>
</tr>
<tr>
<td>SC</td>
<td>Clayey sand</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The USCS is a method of describing a soil and placing it into a category or group which has distinct engineering properties. From the symbol for a classification group or the description, someone trained in the use of the USCS immediately has an idea about the permeability, compressibility, shear strength, and volume change potential of the soil and how it may be affected by water and frost. Also, from the
classification symbols, a contractor can often estimate excavation characteristics, dewatering problems, and workability of a soil.

Soils can be placed into 15 basic groups according to basic engineering properties. Only the material that passes a 3-inch (75-mm) sieve is classified. Each basic group has a distinct two-letter symbol, as shown in table 3.2.1.

<table>
<thead>
<tr>
<th>CLASSIFICATION GROUP SYMBOL</th>
<th>CLASSIFICATION AND DESCRIPTION OF MATERIAL</th>
<th>% PLUS 3 in (BY VOLUME)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SM</td>
<td>0.0 to 3.1 ft SILTY SAND: About 70% coarse to fine, hard, angular sand; about 25% nonplastic fines, rapid dilatancy, no dry strength; about 5% fine, hard, angular gravel; maximum size, 10 mm; moist, brown, faint organic odor; some roots present; easy to auger; no reaction with HCl.</td>
<td>3-5 in 5-12 in PLUS 12 in</td>
</tr>
<tr>
<td>3.1 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GC</td>
<td>3.1 to 6.7 ft CLAYEY GRAVEL: About 75% coarse to fine, hard subrounded gravel; about 15% fines with medium plasticity, high dry strength, medium toughness; about 10% coarse, hard, subrounded sand; maximum size, 75 mm; dry, brown; hard to auger; strong reaction with HCl.</td>
<td></td>
</tr>
<tr>
<td>6.7 ft</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SC</td>
<td>6.7 to 9.8 ft CLAYEY SAND WITH GRAVEL: About 50% coarse to fine, hard, subangular to subrounded sand; about 25% fine, hard, subangular to subrounded gravel; about 25% fines with medium plasticity, high dry strength, medium toughness; maximum size, 20 mm; wet, reddish-brown; easy to auger, weak reaction with HCl.</td>
<td></td>
</tr>
<tr>
<td>9.8 ft</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

REMARKS: Stopped hole due to limit of equipment

Figure 3.2.1.—Typical soil classification log.
The soil classification chart (table 3.2.2) shows how the classification groups are related and how each group is identified.

### Table 3.2.2.—Soil Classification Chart (Laboratory Method).

<table>
<thead>
<tr>
<th>CRITERIA FOR ASSIGNING GROUP SYMBOLS AND GROUP NAMES USING LABORATORY TESTS</th>
<th>SOIL CLASSIFICATION</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>COARSE-GRAINED SOILS</strong></td>
<td><strong>GROUP SYMBOL</strong></td>
</tr>
<tr>
<td>GRAVELS</td>
<td>CLEAN GRAVELS</td>
</tr>
<tr>
<td>More than 50% of coarse fraction retained on No. 4 sieve</td>
<td></td>
</tr>
<tr>
<td>GRAVELS WITH FINES</td>
<td>Fines classify as ML or MH</td>
</tr>
<tr>
<td>More than 12% fines&lt;sup&gt;b&lt;/sup&gt;</td>
<td>Fines classify as CL or CH</td>
</tr>
<tr>
<td><strong>SANDS</strong></td>
<td>CLEAN SANDS</td>
</tr>
<tr>
<td>50% or more of coarse fraction passes No. 4 sieve</td>
<td></td>
</tr>
<tr>
<td>SANDS WITH FINES</td>
<td>Fines classify as ML or MH</td>
</tr>
<tr>
<td>More than 12% fines&lt;sup&gt;b&lt;/sup&gt;</td>
<td>Fines classify as CL or CH</td>
</tr>
<tr>
<td><strong>FINE-GRAINED SOILS</strong></td>
<td><strong>GROUP SYMBOL</strong></td>
</tr>
<tr>
<td>SILTS AND CLAYS</td>
<td><strong>inorganic</strong></td>
</tr>
<tr>
<td>Liquid limit less than 50</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>organic</strong></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>SILTS AND CLAYS</td>
<td><strong>inorganic</strong></td>
</tr>
<tr>
<td>Liquid limit 50 or more</td>
<td></td>
</tr>
<tr>
<td></td>
<td><strong>organic</strong></td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Highly organic soils</strong></td>
<td></td>
</tr>
</tbody>
</table>

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**Appendix 3.2**

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**Note:****

- Based on the material passing the 3-in (75-mm) sieve
- If field sample contained cobbles and/or boulders, add "with cobbles and/or boulders" to group name.
- Gravels with 5 to 12% fines require dual symbols:
  - GW-GM well-graded gravel with silt
  - GW-GC well-graded gravel with clay
  - GP-GM poorly graded gravel with silt
  - GP-GC poorly graded gravel with clay
- Sands with 5 to 12% fines require dual symbols:
  - SW-SM well-graded sand with silt
  - SW-SC well-graded sand with clay
  - SP-SM poorly graded sand with silt
  - SP-SC poorly graded sand with clay
- C = D<sub>s</sub>/D<sub>c</sub> = (D<sub>s</sub><sup>2</sup>)/D<sub>c</sub><sup>2</sup> or D<sub>c</sub><sup>2</sup>/D<sub>s</sub><sup>2</sup>
- If fines classify as CL-ML, use dual symbol GC-GM, SC-SM.
- If fines are organic, add "with organic fines" to group name.
- If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel" whichever is predominant.
- If soil contains 2 to 15% gravel, add "with gravel" to group name.
- If the liquid limit and plasticity index plot in hatched area on plasticity chart, soil is a CL-ML, sily clay.
- If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel" whichever is predominant.
- If soil contains 30% plus No. 200, predominantly sand, add "sandy" to group name.
- If soil contains 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
- PI plots on or above "A" line.
- PI plots below "A" line.
Soil classification is particularly important in the early stages of a construction project. Logs of soil explorations—such as auger holes, drill holes, or test pits—that contain soil classifications and descriptions can be used for:

- Identifying potential foundation problems
- Determining the need for additional field and laboratory investigations
- Informing designers and contractors of the available construction materials
- Making preliminary cost estimates
- Estimating excavation, dewatering, and workability characteristics

**Laboratory Tests**

The laboratory tests necessary for classifying soils are the gradation analysis and the Atterberg limits (or consistency) tests. For USCS, only material smaller than 3 inches in diameter is used for classification.

**Gradation Analysis Test**

In the gradation analysis test, the soil is dried and shaken through a series of sieves containing progressively smaller openings. The mass of soil retained on each sieve is determined, and the percentage (based on dry mass) passing the various opening sizes is plotted on a gradation chart, as shown in figure 3.2.2.

The following terms are used to describe various size fractions of the particles:

**Gravel:** Particles larger than no. 4 sieve (4.75 mm) and smaller than 3 inches.

**Sand:** Particles larger than no. 200 sieve (75 μm) and smaller than no. 4 sieve (4.75 mm).

**Fines:** Particles smaller than no. 200 sieve (75 μm).

The results of a gradation analysis test report the percent of fines, sand, and gravel. For the gradation chart shown, the summary would be: 34 percent gravel, 52 percent sand, and 14 percent fines.
Figure 3.2.2.—Completed gradation analysis test chart.
Atterberg Limits Tests

The physical properties of most fine-grained soils, and particularly clayey soils, are greatly affected by the moisture content. A clay may be very soft like a thick soup, or it may be very hard and dense, depending on its moisture content. Between those extremes, the clay may be molded and formed without cracking or rupturing. In this condition, the clay is referred to as being plastic. Plasticity is an outstanding characteristic of clays and is used to identify and distinguish clayey soils.

In 1911, a Swedish soil scientist, A. Atterberg, developed a series of hand tests for determining the plasticity of soils. These tests are now known as the Atterberg limits or soil consistency tests.

Four states can be recognized for describing the consistency of a soil: (1) the liquid state, (2) the plastic state, (3) the semisolid state, and (4) the solid state. These states of consistency are related to the moisture content. Although the transition between pairs of states is gradual, test conditions have been established arbitrarily to determine the moisture content as a definite point in the transition from one state to another. These moisture contents are called the liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). These tests are performed only on the soil fraction which passes a no. 40 (425 μm) sieve.

The significance of the limits and their relationship to the states of consistency can best be explained by a discussion of figure 3.2.3. As a very wet, fine-grained soil dries, it passes progressively through different states of consistency. In a very wet condition, the soil will act like a viscous liquid; it is in a liquid state. As the soil dries, the volume of the soil mass decreases nearly proportionally to the loss of water. When the moisture in the soil reaches a value equivalent to the liquid limit, the mass enters the plastic state.

![Figure 3.2.3.—Consistency limits.](image-url)
The LL is defined as that moisture content, expressed as a percentage of the dry mass of soil, at which the soil first shows a small but definite shear strength as the moisture content is reduced. Conversely, with increasing moisture, it is that moisture content at which the soil mass just starts to become fluid under the influence of a series of standard shocks.

As the moisture content is reduced below the LL, the soil mass becomes stiffer and will no longer flow as a liquid. However, it will continue to be deformable (plastic), without cracking, until the PL is reached.

The PL is defined as that moisture content, expressed as a percentage of the dry mass of soil, at which the soil mass ceases to be plastic and becomes brittle. The PL is always determined by reducing the moisture content of the soil mass until a 1/8-inch diameter thread of the soil begins to crumble.

The plasticity index (PI) is the numerical algebraic difference between the LL and PL and represents the range of moisture content within which the soil is plastic. Silts have low or no PI's, while clays have higher PI's. The PI, in combination with the LL, indicates how sensitive the soil is to changes in moisture.

As the moisture content of a soil is reduced below the PL, the soil enters a semisolid state; that is, it can be deformed, but considerable force is required to do so, and the soil cracks. As further drying takes place, the soil mass will eventually reach a solid state at which no further shrinkage will occur. The moisture content that would fill the voids in the soil under the solid state condition is the SL. This is the moisture content below which a reduction in moisture will not cause a decrease in the volume of the soil mass.

In summary:

- LL (liquid limit) is the moisture content of a soil at the point at which it changes from a plastic state to a liquid state;
- PL (plastic limit) is the moisture content of a soil at the point at which it changes from a semisolid state to plastic state;
- PI (plasticity index) is the range of moisture within which the soil is plastic (LL-PL); and
- SL (shrinkage limit) is the moisture content of a soil below which a reduction in moisture will not cause a decrease in the volume of the soil mass.

These limits and states are further illustrated in table 3.2.3.
Table 3.2.3.—Atterberg limits.

<table>
<thead>
<tr>
<th>State</th>
<th>Description</th>
<th>Boundary or limit</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid</td>
<td>A slurry; pea soup to viscous butter</td>
<td>Liquid limit (LL)</td>
</tr>
<tr>
<td>Plastic</td>
<td>Soft butter to stiff putty; deforms but will not crack</td>
<td>Plastic limit (PL)</td>
</tr>
<tr>
<td>Semisolid</td>
<td>Cheese; deforms permanently, but cracks</td>
<td>Shrinkage limit (SL)</td>
</tr>
<tr>
<td>Solid</td>
<td>Hard candy; fails completely upon deformation</td>
<td></td>
</tr>
</tbody>
</table>


The three most basic divisions of soil are coarse-grained, fine-grained, and highly organic. Most of the soils in the Western United States fall into the first two divisions. Highly organic soils (symbol PT) are readily identified by color, odor, and spongy feel; they frequently have a fibrous texture.

The first step in classifying a soil is to decide whether it is fine-grained or coarse-grained. The division between fine-grained particles and coarse-grained particles is the no. 200 (75-µm) sieve. All soil particles retained on a no. 200 sieve are coarse-grained; those that pass are fine-grained. The distinction between coarse-grained and fine-grained soils is based on the gradation (or grain-size distribution) of the soil. First, any material over 3 inches (75 mm) is screened or separated out. If the remaining material consists of greater than 50 percent (by mass) coarse-grained particles, the material is coarse-grained; otherwise, it is fine-grained.

**Example 1:**

In a gradation test, particles larger than 3 inches were removed from a soil, and 39 percent of the soil passed through the 75-µm (no. 200) sieve. Therefore, 61 percent of the material is larger than the no. 200 sieve, so it is classified as a coarse-grained soil.
Example 2:

The gradation of a soil resulted in 30 percent gravel, 17 percent sand, and 53 percent fines. The soil is a fine-grained soil because 50 percent or more (53 percent) of the soil is fines (fine-grained particles).

The next step is to determine whether the coarse-grained soil is a sand or a gravel. If at least half of the coarse particles are sand-sized—passing a 1/4-in. (no. 4) sieve but retained on a no. 200 sieve—the soil is classified as a sand. If more than half of the coarse particles are gravel-sized—passing a 3-inch sieve but retained on a 1/4-inch sieve—the soil is a gravel.

If the percent of sand is equal to or greater than the percent of gravel, the soil is a sand. If the percent of gravel is more than the percent of sand, the soil is a gravel.

Example 3:

A soil contains: 43 percent gravel
33 percent sand
24 percent fines

The soil is coarse-grained (76 percent larger than the no. 200 sieve), and the percent gravel is greater than the percent sand, so the soil is a GRAVEL.

Example 4:

A soil contains: 33 percent gravel
43 percent sand
24 percent fines

The soil is course-grained (76 percent larger than the no. 200 sieve), and the percent sand is greater than the percent gravel, so the soil is a SAND.

Inorganic Fine-Grained Soils (CL, ML, CH, MH)

There are four classifications for inorganic fine-grained soils. The group symbols and basic group names are: CL, Lean Clay; CH, Fat Clay; ML, Silt; and MH, Elastic Silt.

The classification of fine-grained soils depends on the values of LL and PI obtained from performing the Atterberg limits tests on the soil. Silts and clays are thus distinguished by their behavior and not their grain size. The Atterberg limits tests are performed on the portion of soil that passes the no. 40 sieve. However, test results are used to classify the fine-grained particles.
The classification of inorganic fine-grained soils is determined by plotting the values of LL versus PI on a plasticity chart. The plasticity chart (shown as figure 3.2.4) has two main division lines. One division line is a diagonal line across the chart termed the “A” line. This line separates clays and silts. If the LL and PI values plot on or above the “A” line, the soil is a clay (C). If the LL and PI values plot below the “A” line, the soil is a silt (M). The other division line is at LL = 50. This line separates silts and clays of low LL’s from silts and clays of high LLW’s. If the LL of the soil is 50 or more, the second letter of the classification symbol is “H.” If the LL is 49 or less, the second letter is “L.” In general, soils with LL values of 50 or more are highly compressible, while soils with LL values of less than 50 are not.

Figure 3.2.4.—Plasticity chart for classifying inorganic fine-grained soils.

The “U” line shown on the plasticity chart is not used for classification, but is useful in evaluating test data.
The plasticity chart is thus divided into four main areas:

- CL—clay with low liquid limit
- ML—silt with low liquid limit
- CH—clay with high liquid limit
- MH—silt with high liquid limit.

Example 5:

The LL and PI values for the following fine-grained soils are plotted on the plasticity chart and would be classified as shown:

(A) \( LL = 38, PI = 22 \)  
    CL—Lean Clay

(B) \( LL = 32, PI = 5 \)  
    ML—Silt

(C) \( LL = 63, PI = 38 \)  
    CH—Fat Clay

(D) \( LL = 73, PI = 12 \)  
    MH—Elastic Silt
Organic soils are soils that would be classified as fine-grained (50 percent or more of the soil passes the no. 200 sieve) but contain enough organic matter to influence the engineering properties of the soil. This is determined by evaluating the effect of the organic matter on the LL of the soil. The LL should be determined on the soil using a wet preparation method; that is, the soil should not be allowed to dry from its natural moisture content. Then a second LL should be performed on soil that has been oven-dried, typically overnight, to a constant mass. The soil is an organic soil if the LL after oven-drying is less than 75 percent of the non-dried LL or

\[
\frac{LL\text{ (oven-dried)}}{LL\text{ (not dried)}} < 0.75
\]

If this criterion is met, then classify the soil as an organic silt, OL or OH, or as an organic clay, OL or OH, by plotting the values of the non-dried (LL, and the PI) (calculated using the non-dried LL) on the plasticity chart shown in figure 3.2.5.

---

**Figure 3.2.5.—Plasticity chart for classifying organic soils.**
If the (LL, PI) value is on or above the "A" line and the PI is 4 or greater, then if the LL is below 50, the soil is an organic clay, OL; if the LL is 50 or greater, the soil is an organic clay, OH.

If the (LL, PI) values plot below the "A" line, the soil is classified as an organic silt, OL, if the LL is below 50, or as an organic silt, OH, if the LL is 50 or greater.
Glossary

**Atterberg Limits.** The boundaries (determined by laboratory tests) of moisture content in a soil between the liquid state and the plastic state (known as the liquid limit), between the plastic state and the semisolid state (known as the plastic limit), and between the semisolid state and the solid state (known as the shrinkage limit).

**Clay.** Soil passing the no. 200 (75-μm) U.S. standard sieve that exhibits plasticity (putty-like properties), with a range of moisture contents and considerable strength when air-dried.

**Coefficient of Curvature** (Cc). The ratio \( D_{30}^2 / (D_{10} \times D_{60}) \) where \( D_{60}, D_{30}, \) and \( D_{10} \) are the particle diameters corresponding to 60, 30, and 10 percent finer on the cumulative gradation curve, respectively.

**Coefficient of Uniformity** (Cu). The ratio \( D_{60} / D_{10} \), where \( D_{60} \) and \( D_{10} \) are the particle diameters corresponding to 60 and 10 percent finer on the cumulative gradation curve, respectively.

**Consistency.** The relative ease with which a soil can be deformed.

**Consistency Limits.** See Atterberg Limits.

**Fines.** Portion of soil finer than a no. 200 (75-μm) U.S. standard sieve.

**Gradation.** The proportions by mass of a soil or fragmented rock distributed in specified particle-size ranges.

**Gravel.** Particles of rock that will pass a 3-inch (75-mm) U.S. standard sieve and be retained on a no. 4 (4.75-mm) U.S. standard sieve.

**Liquid Limit (LL).** The moisture content corresponding to the arbitrary limit between the liquid and plastic states of consistency of a soil.

**Moisture Content (w) in %.** The ratio expressed as a percentage of the mass of water in a given soil mass to the mass of solid particles.

**Organic Clay.** A clay with sufficient organic content to influence the soil properties.

**Organic Silt.** A silt with sufficient organic content to influence the soil properties.

**Organic Soil.** Soil with a high organic content. In general, organic soils are very compressible and have poor load-sustaining properties.

**Particle-Size Analysis.** See Gradation Analysis.

**Plasticity Index (PI).** Numerical difference between the liquid limit and the plastic limit.

**Plastic Limit (PL).** The moisture content corresponding to an arbitrary limit between the plastic and the semisolid states of consistency of a soil.
**Sand.** Particles of rock that will pass the no. 4 (4.75-mm) U.S. standard sieve and be retained on the no. 200 (75-μm) U.S. standard sieve.

**Shrinkage Limits.** The maximum moisture content at which a reduction in moisture content will not cause a decrease in volume of the soil mass.

**Silt.** Material passing the no. 200 (75-μm) U.S. standard sieve that is non-plastic or very slightly plastic and that exhibits little or no strength when air-dried.

**Soil.** Sediments or other unconsolidated accumulations of solid particles produced by the physical and chemical disintegration of rocks, that may or may not contain organic matter.

**Water Content.** See Moisture Content.
APPENDIX 3.3
This appendix describes the Oregon Department of Transportation (ODOT) Rock Classification System and is taken from Oregon Department of Transportation, Highway Division, 1987, *Soil and Rock Classification Manual*: Oregon D.O.T., Salem, OR, 50 p.
ROCK CLASSIFICATION

Rock classification for engineering purposes consists of two basic assessments: that for intact character, such as a hand specimen or small fragment; and in situ character, or engineering features of rock masses (Ref.1):

**Intact character:** classification of the intact rock, such as hand specimens or core, is in terms of its origin, mineralogical makeup, texture, and degree and nature of chemical and physical weathering or alteration.

**In situ character:** classification of in-place rock masses includes the nature and orientation of its constituent interlocking blocks, plates, or wedges formed by bounding discontinuities such as bedding, foliation planes, joints, shear planes, shear zones and faults.

Both assessments are essential for design. Both characteristics are the basis for rock slope design and excavation and many facets of rock anchorage and bearing capacity determinations.

**Rock Name**

Rocks are classically divided into three general categories: igneous, sedimentary and metamorphic.

Igneous rocks are classified based on mineralogy and genetic occurrence (intrusive or extrusive). Texture is the most conspicuous feature of genetic occurrence.

Sedimentary rocks are classified on the basis of grain size, mineralogy and on the relationship between grains.
The most conspicuous features of metamorphic rocks are generally their structural features, especially foliation.

The complete name of a rock specimen or rock unit should include texture and lithologic name. The rock name should be in simple geologic terms. The rock name should be completely written in capital letters. The following tables present common rock names and their characteristics.

### TABLE 16: COMMON IGNEOUS ROCKS

(Ref. 9)

<table>
<thead>
<tr>
<th>Intrusive (coarse-grained)</th>
<th>Essential Minerals</th>
<th>Common Accessory Minerals</th>
<th>Extrusive (fine-grained)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granite</td>
<td>Quartz, K-feldspar</td>
<td>Plagioclase, Mica, Amphibole, Pyroxene</td>
<td>Rhyolite</td>
</tr>
<tr>
<td>Diorite</td>
<td>Plagioclase</td>
<td>Mica, Amphibole, Pyroxene</td>
<td>Andesite</td>
</tr>
<tr>
<td>Gabbro</td>
<td>Plagioclase, Pyroxene</td>
<td>Amphibole</td>
<td>Basalt</td>
</tr>
</tbody>
</table>
TABLE 17: IGNEOUS ROCK TEXTURES

<table>
<thead>
<tr>
<th>Texture</th>
<th>Grain Size</th>
<th>Rock Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pegmatitic</td>
<td>Very large; diameters measured in inches or feet. Wide range of sizes.</td>
<td>Intrusive</td>
</tr>
<tr>
<td>Phaneritic</td>
<td>Can be seen with naked eye</td>
<td>Intrusive or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Extrusive</td>
</tr>
<tr>
<td>Aphanitic</td>
<td>Cannot be seen with naked eye</td>
<td>Extrusive or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Intrusive</td>
</tr>
<tr>
<td>Glassy</td>
<td>No grains present</td>
<td>Extrusive</td>
</tr>
<tr>
<td>Porphyritic</td>
<td>Grains of two widely different sizes</td>
<td>Intrusive and</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Extrusive</td>
</tr>
</tbody>
</table>

TABLE 18: PYROCLASTIC ROCKS

<table>
<thead>
<tr>
<th>ROCK NAME</th>
<th>CHARACTERISTICS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cinders</td>
<td>Uncemented glassy and vesicular ejecta 4-32 mm size</td>
</tr>
<tr>
<td>Tuff Breccia</td>
<td>Composed of ejecta &gt;32 mm size, in ash/tuff matrix, indurated</td>
</tr>
<tr>
<td>(agglomerate)</td>
<td></td>
</tr>
<tr>
<td>Lapilli Tuff</td>
<td>Composed of ejecta 4-32 mm size, in ash/tuff matrix, indurated</td>
</tr>
<tr>
<td>Tuff</td>
<td>Cemented volcanic ash particles &lt;4 mm size, indurated</td>
</tr>
<tr>
<td>Pumice</td>
<td>Excessively vesiculated glassy lava</td>
</tr>
</tbody>
</table>
A. Mechanical Sedimentary Rocks

<table>
<thead>
<tr>
<th>Rock Name</th>
<th>Original Sediment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conglomerate</td>
<td>Gravel, or sand and gravel</td>
</tr>
<tr>
<td>Sandstone</td>
<td>Sand</td>
</tr>
<tr>
<td>Siltstone</td>
<td>Silt</td>
</tr>
<tr>
<td>Claystone</td>
<td>Clay</td>
</tr>
<tr>
<td>Mudstone</td>
<td>Silt, clay, possibly with sand and/or gravel inclusions, nonoriented</td>
</tr>
<tr>
<td>Shale (laminated claystone/siltstone)</td>
<td>Oriented, laminated, fissile, clay and silt</td>
</tr>
</tbody>
</table>

B. Chemical Sedimentary Rocks

<table>
<thead>
<tr>
<th>Rock Name</th>
<th>Main Mineral</th>
</tr>
</thead>
<tbody>
<tr>
<td>Limestone</td>
<td>Calcite</td>
</tr>
<tr>
<td>Dolomite</td>
<td>Dolomite</td>
</tr>
<tr>
<td>Chert</td>
<td>Quartz</td>
</tr>
</tbody>
</table>

A modifier may be necessary to describe a sedimentary rock formed from a combination of different soil types, i.e., a "silty SANDSTONE" would be predominantly composed of sand grains with a lesser amount of silt grains. This distinction is only necessary when the modifier has engineering significance. The term mudstone could be used when the composition of the sedimentary rock is uncertain or variable.
### A. Foliated Metamorphic Rocks

<table>
<thead>
<tr>
<th>Rock Name</th>
<th>Texture</th>
<th>Formed From</th>
<th>Main Minerals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slate</td>
<td>Platy, fine-grained</td>
<td>Shale</td>
<td>Mica, quartz</td>
</tr>
<tr>
<td>Schist</td>
<td>Irregular layers, medium-grained</td>
<td>Slate, igneous rocks</td>
<td>Mica, quartz, feldspar, amphibole</td>
</tr>
<tr>
<td>Gneiss</td>
<td>Layered, coarse-grained</td>
<td>Igneous rocks, schist, sandstone</td>
<td>Mica, quartz, feldspar, amphibole</td>
</tr>
</tbody>
</table>

### B. Nonfoliated Metamorphic Rocks

<table>
<thead>
<tr>
<th>Rock Name</th>
<th>Texture</th>
<th>Formed From</th>
<th>Main Minerals</th>
</tr>
</thead>
<tbody>
<tr>
<td>Marble</td>
<td>Crystalline</td>
<td>Limestone, dolomite</td>
<td>Calcite dolomite</td>
</tr>
<tr>
<td>Quartzite</td>
<td>Crystalline</td>
<td>Sandstone</td>
<td>Quartz</td>
</tr>
<tr>
<td>Serpentinite</td>
<td>Massive to layered, fine to coarse-grained</td>
<td>Ultramafic rocks, i.e., peridotite, gabbro</td>
<td>Serpentine</td>
</tr>
</tbody>
</table>
**Vesicularity** Vesicles in volcanic rocks are rounded cavities due to gas bubbles in molten lava. Cavities or openings in other rocks (e.g., intergranular space) should be described in other terms, such as porosity (e.g., porous sandstone).

The occurrence of vesicles are to be reported using the Comparison Chart (Figure 5) to estimate relative percent area occupied by vesicles and the designations in Table 21.

**FIGURE 5: DEGREE OF VESICULARITY COMPARISON CHART (PERCENT BY VOLUME)**

(Ref. 19)

![Comparison Chart](image)

**TABLE 21: DEGREE OF VESICULARITY**

<table>
<thead>
<tr>
<th>Term</th>
<th>Percentage (by volume) of Total Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>Some Vesicles</td>
<td>5 - 25%</td>
</tr>
<tr>
<td>Highly Vesicular</td>
<td>15 - 50%</td>
</tr>
<tr>
<td>Scoriaceous</td>
<td>&gt; 50%</td>
</tr>
</tbody>
</table>
Color

Rock color is not in itself a specific engineering property, but may be an indicator of the influence of other significant conditions such as groundwater (e.g., mottling indicating wet/dry cycles), and alteration/weathering. Color may also be an aid in subsurface correlation.

The color should be determined from fresh samples. Describe the "net" color of the rock mass. Wetting the rock sample may be necessary if drying has occurred. Use common colors (such as white, yellow, red, brown, green, gray, blue, or black) because they are basic and easier for others to identify. For variations use hyphenated combinations such as red-brown. Do not use "ish", or unusual colors. For example: instead of turquoise or aqua, use green-blue or blue-green, respectively. Avoid listing more than two colors.

Degree of Weathering

Weathering and alteration should be described as part of the rock classification. Weathering is the process of mechanical and/or chemical breakdown of rocks through exposure to the elements, which include rain, wind, plant action, groundwater, ice, and changes of temperature. In general, the strength of rock tends to decrease as the degree of weathering increases. In the earliest stages, weathering is manifested by discoloration of intact rock and only slight changes in rock texture. With time, significant changes in rock hardness, strength, compressibility and permeability occur, and the rock mass is altered until the rock is decomposed to soil. For determining stages of weathering for rock, use Table 22, Scale of Relative Rock Weathering. For example, a basalt that is more than 50 percent decomposed (but not completely) would be described as: "BASALT; predominantly decomposed." The degree of weathering should be determined for each rock core sample; multiple designations would be required for variable rock conditions.
In select cases, the term alteration may be used, which applies specifically to changes in the chemical or mineral composition of rock due to hydrothermal or metamorphic activity. Alteration may occur as zones and pockets and can be found at depths far below that of normal rock weathering. Separate the terms weathering and alteration, since alteration does not strictly infer a reduction in rock strength. For example, a gray basalt that is closely jointed with extensive hydrothermal alteration and secondary mineralization, may exhibit only slight weathering along joint surfaces and would be described as: "BASALT; gray; slightly weathered; close jointed; extensive hydrothermal alteration with secondary mineralization".

**TABLE 22: SCALE OF RELATIVE ROCK WEATHERING**
(Modified, After Ref.3,11)

<table>
<thead>
<tr>
<th>Designation</th>
<th>Field Identification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh</td>
<td>Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.</td>
</tr>
<tr>
<td>Slightly Weathered</td>
<td>Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.</td>
</tr>
<tr>
<td>Moderately Weathered</td>
<td>Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.</td>
</tr>
<tr>
<td>Predominantly Decomposed</td>
<td>Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.</td>
</tr>
<tr>
<td>Decomposed</td>
<td>Rock mass is completely decomposed. Original rock &quot;fabric&quot; may be evident. May be reduced to soil with hand pressure.</td>
</tr>
</tbody>
</table>
Relative Hardness of Rock

Differentiating between rock and soil, for engineering purposes, is based primarily on values of unconfined compressive strength. Rock hardness is a measure of rock strength, and is controlled by many factors including degree of induration, cementation, crystal bonding, and/or degree of weathering. Determination of rock hardness may be estimated through manual field tests yielding a "field classification", which can be refined through further field and laboratory testing. The scale of rock hardness to be used is presented on Table 23. The relative hardness of rock should be determined for each rock core sample: multiple designations would be required for variable rock conditions, such as changes in weathering and joint filling.

TABLE 23: SCALE OF RELATIVE ROCK HARDNESS
(Modified, After Ref.3,12,17)

<table>
<thead>
<tr>
<th>Term</th>
<th>Hardness Designation</th>
<th>Field Identification</th>
<th>Approximate Unconfined Compressive Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extremely Soft</td>
<td>R0</td>
<td>Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.</td>
<td>&lt; 100 psi</td>
</tr>
<tr>
<td>Very Soft</td>
<td>R1</td>
<td>Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife. Scratched with finger nail.</td>
<td>100-1000 psi</td>
</tr>
<tr>
<td>Soft</td>
<td>R2</td>
<td>Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.</td>
<td>1000-4000 psi</td>
</tr>
<tr>
<td>Medium Hard</td>
<td>R3</td>
<td>Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.</td>
<td>4000-8000 psi</td>
</tr>
<tr>
<td>Hard</td>
<td>R4</td>
<td>Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.</td>
<td>8000-16000 psi</td>
</tr>
<tr>
<td>Very Hard</td>
<td>R5</td>
<td>Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.</td>
<td>&gt; 16000 psi</td>
</tr>
</tbody>
</table>
Structure

Structure refers to large-scale (megascopic) planar or oriented features which are significant to the overall strength, permeability, and breakage characteristics of the rock unit. Planar structural features include joints, bedding, and faults. These terms are defined below. Other oriented structural features include mineral/grain orientation (i.e., foliation, flow banding and folded originally planar features) or root holes.

**Joints** Planar breaks or fractures in rock along which no movement has occurred parallel to the fracture surface are defined as joints. They may range from perpendicular to parallel in orientation with respect to bedding. Repetitive patterns of more or less parallel joints is called a joint set. Two or more joint sets or a pattern of joints define a joint system. The number of joint sets is most reliably obtained from rock exposures.

**Stratification** Stratification of rock is evidenced by changes in texture, composition, age or unique forms. Bedding applies primarily to sedimentary and pyroclastic rocks. Other terms related to stratification are defined in Table 24.

<table>
<thead>
<tr>
<th>TERM</th>
<th>CHARACTERISTICS</th>
</tr>
</thead>
<tbody>
<tr>
<td>laminations</td>
<td>thin beds (&lt; 1 cm.)</td>
</tr>
<tr>
<td>fissile</td>
<td>tendency to break along laminations</td>
</tr>
<tr>
<td>parting</td>
<td>tendency to break parallel to bedding, any scale</td>
</tr>
<tr>
<td>foliation</td>
<td>non-depositional, e.g., segregation and layering of minerals in metamorphic rocks</td>
</tr>
</tbody>
</table>
Joint or Bedding Spacing  In determining the range of distances between individual joints or beds, care must be taken to distinguish between joints and mechanical breaks that are caused by handling or drilling. These types of mechanical breaks are typically rough and irregular, showing a fresh rock surface and are disregarded for description. Some mechanical breaks, though, may be caused by handling or drilling, but occur along existing joints or fractures, and should be described accordingly. Joint/bedding spacing is based on Table 25.

<table>
<thead>
<tr>
<th>Spacing</th>
<th>Joint spacing terms</th>
<th>Bedding/Foliation spacing terms</th>
</tr>
</thead>
<tbody>
<tr>
<td>Less than 2 in.</td>
<td>Very close</td>
<td>Very thin (laminated)</td>
</tr>
<tr>
<td>2 in. - 1 ft.</td>
<td>Close</td>
<td>Thin</td>
</tr>
<tr>
<td>1 ft. - 3 ft.</td>
<td>Moderately close</td>
<td>Medium</td>
</tr>
<tr>
<td>3 ft. - 10 ft.</td>
<td>Wide</td>
<td>Thick</td>
</tr>
<tr>
<td>More than 10 ft.</td>
<td>Very wide</td>
<td>Very thick (massive)</td>
</tr>
</tbody>
</table>

Faults Planar breaks or fractures, along which displacement has occurred parallel to the fracture surface are termed as faults. The presence of gouge (pulverized rock), bedding offset, and/or slickensided surfaces (commonly with mineral or clay coating), may be indicators of fault movement. However, not all slickensides are caused by faulting: slickensides can be caused by deformation (i.e., folds, flows) or landsliding.
**Attitude** The inclination of a joint, fault, or bedding plane or other linear feature is measured from horizontal. Figure 6 presents joint features that should be identified/measured. The angle that striations (slickensides) make with a horizontal line is known as the "rake", as shown on Figure 6.

**FIGURE 6: MEASUREMENT OF JOINTS**

(After Ref. 13)

Strike and dip of joint and bedding planes are usually measured in test pits or on outcrops, since core obtained in most drilling operations will not be properly oriented. Joint and bedding planes should be described in terms of orientation, i.e., strike and dip. Primary and secondary joint sets should be defined where possible and appropriate. Typically in rock, one joint set may yield slabs, two intersecting joint sets may yield wedges, and three or more intersecting joint sets may yield blocks or highly fragmented rock.
**Separation** The separation or relative openness of joints may be described as:

a) **open**, an existing planar surface that is separated or separates easily when handled, and may have mineralization or staining/weathering on the joint surfaces. Where measurable, identify the opening width (aperture). Open joints are possible groundwater drainage paths.

b) **closed**, an existing planar surface that separates with greater difficulty, seen as a "hairline" trace on the outside of the sample/core, and usually does not have soil or mineral surface coating.

c) **healed**, breaks open easily or with difficulty, seen either as a hairline trace or seam of some thickness on the outside of the sample/core, and usually contains soil or minerals as a filling between joint surfaces.

**Filling** This term refers to the material in the space between adjacent surfaces of a discontinuity. The filling material may consist of weathered or hydrothermally altered products, secondary mineral precipitates, mylonite or gouge. The material description and thickness of the filling material should be reported.
Continuity

Continuity is an expression of the lateral extension of the discontinuity, as measured or projected along its strike and dip. Continuity is a very important property of the rock mass, as a single continuous joint may actually control the behavior of the entire mass. Whether or not joints are continuous may require test pit, outcrop, or additional borehole information for confirmation. Description of joint continuity, as defined in Table 26, should include an indication of certainty and the method of observation.

TABLE 26: DEGREE OF CONTINUITY
(Ref. 12)

<table>
<thead>
<tr>
<th>Term</th>
<th>Length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Discontinuous</td>
<td>0 - 5 ft.</td>
</tr>
<tr>
<td>Slightly continuous</td>
<td>5 - 10 ft.</td>
</tr>
<tr>
<td>Continuous</td>
<td>10 - 40 ft.</td>
</tr>
<tr>
<td>Highly continuous</td>
<td>&gt; 40 ft.</td>
</tr>
</tbody>
</table>

Core Recovery and Rock Quality Designation (ROD)

Core recovery and the Rock Quality Designation are measured indicators of the quality and structure of rock. Both the percent core recovery and the RQD should be determined and recorded on the field boring log for each core run. The core recovery is calculated by dividing the length of core retained (recovered) in the core barrel by the total run length expressed as a percent.
The RQD provides a subjective estimate of rock mass quality/structure. The RQD is a modified core recovery percentage in which only pieces of intact rock core 4 inches or greater in length are measured (average length). The smaller pieces are considered to be the result of close jointing, fracturing or weathering in the rock mass, and are therefore excluded from the RQD determination. The RQD is defined as the cumulative total length of all pieces 4 inches long or longer divided by the total run length, expressed as a percentage. Mechanical breaks, such as caused by handling or drilling, should be noted as such and not included in the RQD calculations.

In some cases, where significant soil is encountered at one end of the core run, the RQD should be determined on the basis of rock core length recovered: where this is done it should be clearly defined. RQD is not applicable to fissile rocks such as shales. Difficulties such as distinguishing natural fractures in the rock core from mechanical breaks and the insensitivity of the RQD to the tightness of individual joints may limit the use of the RQD in evaluating in situ rock properties.

Other Rock Characteristics

Other physical characteristics should be described, depending on the scope and objectives of the project. These may include the following:

Mineralization Secondary mineralization is the introduction of new minerals to a rock mass from an outside source, or through alteration of existing minerals. Mineralization may occur in voids, along joints or within the ground mass.
Iron-oxide staining usually indicates the static groundwater level may fluctuate within the discolored zone. The iron oxide may only be a discoloration of surfaces, or an accumulation of bright orange material several inches thick and varying in hardness. Sulfide or carbonate minerals, such as pyrite or calcite, may be present and could denote groundwater of high mineral or bicarbonate content. Alteration products may indicate an increase in hardness/brittleness (i.e., silicification, usually due to hydrothermal alteration), or reduction of rock strength if soft clay minerals have developed along joints or replaced major constituent minerals (e.g., the feldspar crystals in basalt altered to clay).

**Slaking** The tendency for rock to disintegrate under conditions of wetting and drying, or when exposed to air is called slaking. This behavior is related primarily to the chemical composition of the material. It may be identified in the field if samples shrink and crack, or otherwise degrade upon drying, or being exposed to the air for several hours. If degradation occurs, and slaking is suspected, an air-dried sample may be placed in clean water to observe a reaction. The greater the tendency for slaking, the more rapidly degradation will occur. This tendency should be expressed on field logs as "potential for slaking", and can be confirmed through laboratory testing.

**Field Unit Weight** The unit weight of rock can be important and useful in engineering design and practice. The unit weight can be determined by performing a field bulk specific gravity test and multiplying by the unit weight of water to get the rock unit weight. The procedure consists of weighing the sample in air (B) and then weighing it in water (C).

\[
\text{field unit weight} = \frac{B}{B - C} \times 62.4, \text{lbs. per. cu. ft.}
\]
**Discontinuity Surface Condition** If applicable to the project, the joint/fault surfaces should be inspected and the surface condition described. Joint surface roughness can be defined in terms of a Joint Roughness Coefficient (JRC), which requires estimation or measurement of the surface unevenness, i.e., rough or smooth undulating, rough or smooth nearly planar. The JRC should be determined in the direction of anticipated block movement. Surface roughness is best determined on in-place discontinuities rather than core samples. For further detail, see References 5, 6, and 13.

**Voids** Open spaces in sedimentary and metamorphic rock are generally caused by chemical dissolution or the action of running water. Since most of these voids result from the action of groundwater, the openings are usually elongate in the horizontal plane. The size of voids, where significant, should be measured and recorded with the rock classification.

**Formation Name**

Various rock units are generally known by formational names (i.e., Columbia River Basalt Formation, Astoria Formation, Umpqua Formation) and can be identified within project boundaries by examination of core samples, rock outcrops, and geologic literature. Where the formation name is known, it should be included at the end of the rock classification (in parentheses).
REFERENCES

1. AASHTO, Manual on Foundation Investigations, 1978


15. Ontario Ministry of Transportation and Communications, Soil Classification Manual, Ontario, Canada, 1980


Appendix 3.3
APPENDIX 3.4
3.4 Unified Rock Classification System

Unified Rock Classification System

DOUGLAS A. WILLIAMSON, Senior Engineering Geologist
U.S. Department of Agriculture, Willamette National Forest, P.O. Box 10607, Eugene, OR 97440

ABSTRACT

The Unified Rock Classification System (URCS) provides a reliable and rapid method of communicating detailed information about rock conditions pertinent to design and construction of civil engineering projects. The URCS consists of four fundamental physical properties: 1) weathering, 2) strength, 3) discontinuities and 4) density. A general assessment of rock performance is then based on a grouping of the four key elements to aid in making engineering judgments. These individual properties are estimated in the field with the use of a hand lens, a 1-pound ballpeen hammer, a spring-loaded "fish" scale and a bucket of water. Each property is divided into five ratings which convey uniform meaning to engineering geologists, design engineers, inspectors and contractors as well as contract appeal board members.

Subjective terminology, such as "slightly weathered, moderately hard, highly fractured and lightweight," varies widely in meaning, depends on individual and professional experience, and cannot be quantified with any reliability. The URCS is not intended to supplant existing geologic classifications but it does offer a suitable alternative to ambiguous descriptive terminology.

INTRODUCTION

The Unified Rock Classification System (URCS) allows rapid preliminary assessments of rock conditions by simple field tests that establish basic engineering properties. The URCS is useful for all earth-associated design and construction projects. Pertinent natural conditions related to design parameters are documented in a convenient manner which can be understood at a glance.

The URCS is engineering shorthand which can be used to convey maximum information. With the URCS, rock is classified in simple, easily understood terms that convey evidence of strength and behavior. The URCS terms convey uniform meaning to
reliable rock information that resulted in successful
design and construction as well as post-construction
evaluations. The URCS in its present form dates
from 1975 and is used by the U.S. Forest Service
in Region 6 and parts of Regions 1 and 5. It has
been found to be a reliable method of communicat-
ing rock conditions (including those in quarries,
retaining walls, and extensive rock excavations) for
the design of forest access roads.

PURPOSE AND NEED

The purpose of the URCS is to establish a means
of making rapid initial assessments of rock condi-
tions related to design and construction by simple
field tests that permit direct estimation of natural
strength parameters. The purpose of this report is
threefold: 1) to present a rock classification for use
in engineering geology and geotechnical investiga-
tions, 2) to outline field procedures that require sim-
ple equipment, and 3) to establish the relationship
between the classification and design and perform-
ance.

Experienced professionals who deal with rock can,
and often do, apply the principles of rock mechanics
without any formally accepted rock classification.
Organizations comprised of employees of many ex-
perience levels commonly have a designated rock
classification system. Rock information is frequent-
ly collected in the field by geologists but utilized in
design by engineers. The URCS is intended to sup-
plement, not supplant, existing geologic classifica-
tions; its specific goal is to provide a means by which
the inherent confusion of subjective terminology can
be eliminated when applied in civil engineering.

Classification is not the chief aim of geotechnical
investigations, but a uniform working classification
is necessary to effectively supply the needs of a large
organization of diverse professional disciplines. The
assertion that there is no need for another classifi-
cation is easily discounted with the statement that
a classification is always needed until one is found
that meets general approval and acceptance and is
used.

The following statements are unfortunately still
ture: “There are as many classifications as there are
gologists,” and “No two geotechnical investigators
will give the same name to the same rock.” Because
of the number of geotechnical personnel working in
the field, it is vital that some uniformity of data exist.
Even now, when one reads geotechnical re-
ports, drilling logs, or contract documents, it is not
possible to be sure that two different geotechnical
specialists who are discussing the same rock are de-
scribing sufficiently identical design characteristics.

A working classification requires uniform symbols,
abbreviations, notations, and definitions that are
established to be acceptable procedures.

The URCS fulfills basic needs of any classifica-
tion:
1) Definitions are developed by simple field tests.
2) Information is presented in simple, understand-
able, non-technical terms.
3) Field conditions are related to design and con-
struction.
4) Notations are flexible to scale of sample, outcrop
or large excavation area and are appropriate to
evaluation.
5) Collected data are verifiable, reproducible and
independent of experience but not training.
6) The system is useful to all levels of experience.
7) The system allows immediate assessment, both
directly and on notes or documents.

BASIC ELEMENTS

The URCS consists of four basic elements which
are major physical properties of rock material and
are related to design and construction. The elements
are: 1) degree of weathering, 2) estimated strength,
3) discontinuities or directional weaknesses, and 4)
unit weight or density.

By establishing limiting values of these four basic
elements by using uniform field tests and observa-
tions, terminology, notations, and abbreviations, the
URCS provides a means for recording and commu-
icating reliable indications of rock properties
and performance. The URCS permits a useful es-
timate of compressive strength, permeability and
shear strength—the three primary properties of rock
masses. When combined with other geotechnical
information (stress history and water table location),
the URCS permits an estimate of rock performance
such as foundation suitability, excavation methods,
slope stability, material use, blasting character and
water transmittal.

The equipment used for the field tests and observa-
tions is simple and available: one’s fingers, a 10-
power hand lens, a 1-pound (0.5-kg) ballpeen ham-
er, a spring-loaded “fish” scale of the 10-pound
(5 kg) range, and a bucket of water. Fingers are used
in determining the degree of weathering and the
lower range of strength. The hand lens is used in
defining the degree of weathering. The ballpeen
hammer is used to estimate the range of unconfined
WILLIAMSON—UNIFIED ROCK CLASSIFICATION

DEGREE OF WEATHERING

<table>
<thead>
<tr>
<th>REPRESENTATIVE</th>
<th>ALTERED</th>
<th>WEATHERED</th>
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</thead>
<tbody>
<tr>
<td>Micro Fresh State (MFS)</td>
<td>Visually Fresh State (VFS)</td>
<td>Stained State (STS)</td>
</tr>
<tr>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>UNIT WEIGHT</td>
<td>RELATIVE ABSORPTION</td>
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<td></td>
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<tr>
<td>COMPARE TO FRESH STATE</td>
<td>NON-PLASTIC</td>
<td>PLASTIC</td>
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ESTIMATED STRENGTH

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<tr>
<th>REACTION TO IMPACT OF 1 LB. BALLPEEN HAMMER</th>
<th>REMOLDING</th>
</tr>
</thead>
<tbody>
<tr>
<td>&quot;Rebounds&quot; (Elastic) (RQ)</td>
<td>&quot;Pits&quot; (Tensional) (PQ)</td>
</tr>
<tr>
<td>&quot;Dents&quot; (Compression) (DQ)</td>
<td>&quot;Craters&quot; (Shears) (CQ)</td>
</tr>
<tr>
<td>&quot;Rebounds&quot; (&gt;15000 psi², &gt;103 MPa)</td>
<td>&quot;Pits&quot; (8000-15000 psi², 55-103 MPa)</td>
</tr>
<tr>
<td>&quot;Dents&quot; (3000-8000 psi², 21-55 MPa)</td>
<td>&quot;Craters&quot; (1000-3000 psi², 7-21 MPa)</td>
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<tr>
<td>Moldable (Friable) (MQ)</td>
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DISCONTINUITIES

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<td>Solid (Random Breakage) (SRB)</td>
<td>Solid (Preferred Breakage) (SPB)</td>
</tr>
<tr>
<td>Solid (Latent Planes Of Separation) (LPS)</td>
<td>Nonintersecting Open Planes</td>
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<tr>
<td>(2-D)</td>
<td>Intersecting Open Planes</td>
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<tr>
<td>(3-D)</td>
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<tr>
<td>ATTITUDE</td>
<td>INTERLOCK</td>
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UNIT WEIGHT

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<tr>
<td>160 pcf</td>
<td>2.55 g/cc</td>
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<td>2.40-2.55 g/cc</td>
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<tr>
<td>130 pcf</td>
<td>2.10 g/cc</td>
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DESIGN NOTATION

WEATHERING

WEIGHT

STRENGTH

DISCONTINUITY

Figure 1. Basic elements of the unified rock classification system.
compressive strength from impact reaction. The spring-loaded scale and bucket of water are used to measure the weight of samples for determining apparent specific gravity.

The URCS design notation consists of underlined groups of four letters ranging from \(A\) through \(E\) which represent the five categories or design-limiting conditions of each of the basic elements of the system. The limiting conditions of the basic elements are described below in the order that they appear in the symbol.

Degree of Weathering

The degree of weathering in the URCS is restricted to chemical processes. The five design-limiting conditions of weathering are:

1) micro fresh state (MFS) designated by \(A\),
2) visually fresh state (VFS) designated by \(B\),
3) stained state (STS) designated by \(C\),
4) partly decomposed state (PDS) designated by \(D\), and
5) completely decomposed state (CDS) designated by \(E\).

These five weathering states are listed in the top line of Figure 1.

Micro Fresh State (MFS)

MFS is determined in the field by examining rock samples with the aid of a 10-power hand lens. This condition is characterized by absence of oxidation alteration of any mineral components. MFS will generally apply only to crystalline rocks, some volcanic rocks, and chemical sedimentary rocks. Investigations of crushed rock and concrete aggregate sources may require MFS rock, but ordinary rock-design evaluations usually do not require such high quality rock.

Visually Fresh State (VFS)

VFS is determined in the field by examining rock samples with the unaided eye. This condition is characterized by a uniform color of the rock material. VFS will apply to all rock types including some clastic sedimentary rocks. VFS samples commonly exhibit maximum unit weight, maximum specimen strength and least water absorption for the site from which comparisons to STS are made. The VFS is generally representative of the standard quality acceptable for all foundation and excavation designs.

Stained State (STS)

STS is determined in the field by examining rock samples with the unaided eye. This condition is characterized by partial or complete discoloration due to oxidation alteration of mineral components, but the specimen cannot be remolded with finger pressure. STS will apply to all rock types and commonly is the highest weathering state of Cenozoic clastic sedimentary rocks. STS specimen strength may or may not vary from that of VFS specimens; unit weights are usually lower and water absorptions are usually higher than VFS specimens.

Partly Decomposed State (PDS)

PDS is determined in the field by applying finger pressure to discolored specimens. The material is solid rock when in place but can be disaggregated into gravel or larger size rock fragments in a matrix of soil. Decomposed granite is an example of PDS weathering. The relative percentage of rock fragments is estimated and the quality of individual fragments is assessed with URCS. The soil fines are determined to be plastic or nonplastic.

The in-place strength is estimated by manual consistency values or by size, shape, and gradation of the remolded aggregate (Terzaghi and Peck, 1948). The remolded aggregate is tested for dilatency, dry strength, and toughness and classified according to field procedures of the Unified Soil Classification System (USCS) (Casagrande, 1948).

Completely Decomposed State (CDS)

CDS is determined in the field by applying finger pressure to discolored specimens in a manner similar to that for PDS specimens. CDS specimens dis-aggregate or remold to soil without gravel or larger size fragments of intact rock. The remolded material is determined to be plastic or nonplastic, and dry strength, dilatency, and toughness tests are performed. The in-place strength is estimated by manual consistency values. Both URCS and USCS symbols are recorded. Note that the URCS boundary separating rock from soil is the No. 4 sieve which is the gravel/sand division. Most investigators, as well as lay persons, generally accept that gravel is composed of rock fragments but sand is composed of mineral grains.

Estimated Strength

A reasonable estimate of specimen strength can be made by striking a sample, piece of rock core, or outcrop with the round end of a ballpeen hammer (or with the rounded head end of a 20-penny nail if the specimen is to be preserved). The resulting characteristic impact reaction indicates a range of unconfined compressive strength (Williamson, 284 Appendix 3.4
The rock specimen or outcrop is struck several times to permit evaluation of uniformity of response, and a quality is assigned based on the distinct reaction. The five kinds of reaction are illustrated in Figure 2.

The reaction of a rock specimen to the impact of a ballpeen hammer is distinct and characteristic depending on the range of unconfined compressive strength. The nature of the reaction, not the magnitude of the reaction is used to assign a strength quality to the specimen. Therefore, the reaction is independent of the intensity of the blow within the limitations of the tool used and the investigator's strength.

The five design-limiting conditions of strength based on impact reaction are: 1) rebound quality (RQ) designated by A, 2) pit quality (PQ) designated by B, 3) dent quality (DQ) designated by C, 4) crater quality (CQ) designated by D, and 5) moldable quality (MQ) designated by E. These five strength states are listed in the second line of Figure 1. The range of unconfined compressive strength in terms of pounds per square inch (psi) and megapascals (MPa) for each of the strength states is also listed in Figure 1.

Rebound Quality (RQ)

RQ rock material shows no reaction under the point of impact and is a true brittle-elastic substance in a mechanical sense. The estimated unconfined compressive strength of RQ material is greater than 15,000 psi (103 MPa). The exact strength value is seldom significant in typical civil engineering applications once the strength reaches this magnitude. RQ rock material produces free-draining fill that is suitable for road aggregate; however, it is often sharp and angular due to its brittleness and therefore may produce a less desirable material than PQ material. RQ rock material has a very high energy transfer in response to blasting and may be difficult to drill and break in the absence of planar separations.

Pit Quality (PQ)

PQ rock material produces a shallow rough pit under the point of impact due to explosive departure of mineral grains. This quality of specimen has an estimated unconfined compressive strength ranging from 8,000 to 15,000 psi (55 to 103 MPa) and is considered to be hard rock by the construction industry. PQ rock material produces free-draining fill and is suitable for road surfacing material. It has high energy transfer in response to blasting which produces generally good fragmentation and satisfactory excavation slopes. No special blasting design procedure is generally necessary.

Dent Quality (DQ)

DQ rock material produces a dent or depression under the point of impact indicating the presence of pore spaces between mineral grains. This quality of specimen has an estimated unconfined compressive strength ranging from 3,000 to 8,000 psi (21 to 55 MPa) and is approximately equivalent to the strength range of concrete. DQ rock material usually does not meet absorption specifications for road aggregate and has a relatively low energy transfer in response to blasting. Special blasting design may be necessary to avoid creating over-size blocks. DQ material is usually not suitable for road fill or surfacing and is not free-draining.

Crater Quality (CQ)

CQ rock material produces a shearing and upthrusting of mineral grains surrounding the point of impact resulting in a depression which resembles a moon crater. This quality of specimen has an estimated unconfined compressive strength ranging from 1,000 to 3,000 psi (7 to 21 MPa). CQ material can usually be recovered during diamond-core drilling operations, has high absorption, and will respond to freeze-thaw stresses by at least cracking and checking. It has very low energy transfer when blasted and can be excavated by means of machinery, produces poorly drained embankments and is not suitable for road fill or surfacing material.

Moldable Quality (MQ)

MQ rock is in a condition which can be molded by finger pressure but retains the fabric of intact rock. The unconfined compressive strength for this
Discontinuities

Directional weaknesses of a rock mass or rock material are termed planar or linear features. Planar separations are open separations that already exist in the rock mass and are defined by relative capacity to transmit water. Linear features are directional weaknesses due to visible or nonvisible alignments of mineral grains in an otherwise solid rock mass or material that usually require blasting or mechanical crushing to produce a separation.

For purposes of design evaluations, linear features are defined by breakage characteristics. Planar features or open planes of separation are defined by the scale dimension of the rock mass examined and by the geometric determination that defines a plane or shape.

The five design-limiting conditions of discontinuities are: 1) solid random breakage (SRB) designated by A, 2) solid preferred breakage (SPB) designated by B, 3) latent planar separations (LPS) designated by C, 4) two-dimensional open planar separations (2-D) designated by D, and 5) three-dimensional open planar separations (3-D) designated by E. These five discontinuity states are listed in the third line of Figure 1.

Solid Random Breakage (SRB)

SRB represents ideal design conditions in which planar and linear features have no effect within the dimensions of the rock mass examined. The specimen strength equals the mass strength so that the strength value of any individual sample tested is directly representative of the entire rock-mass strength. Needless to say, this is seldom the case, except in very limited foundation dimensions.

Solid Preferred Breakage (SPB)

SPB indicates that a nonvisible alignment of mineral grains has resulted in a directional weakness in the rock mass or material. The rock breaks consistently along a uniform angle or direction. SPB rock material may produce an undesirable shape or size for rock aggregate or may prevent the achievement of a desired slope in an excavation. It may be an adverse quality in the production of dimension stone.

Latent Planar Separations (LPS)

LPS indicates a visible alignment of mineral grains or infilling material which may or may not affect the strength or breakage character of the rock mass or material during excavation or crushing. The latent planes may be stronger or weaker than the rock mass. The reaction of LPS material to impact defines the strength estimate. Latent planes occur in patterns or at random and are continuous or discontinuous; the planes may be of measurable thickness. In all cases, the infilling material in the latent plane of separation has an unconfined compressive strength greater than 1,000 psi (7 MPa).

LPS material is usually not a foundation design consideration because the material is, for practical purposes, a solid. In consideration of rock slope design or road aggregate source, blasting energy will, in most cases, be reflected by the latent plane and produce a separation and breakage at right angles to the plane alignment.

Two-Dimensional Open Planar Separations (2-D)

The 2-D category indicates the presence of one or more parallel open planes of separation that pass through the rock mass at the point of examination. The 2-D planar separations may vary in frequency and spacing but do not intersect. The attitude, relief, and continuity of the plane or planes are fundamental elements of design analysis. Water transmission along the open planes can be determined by observations of the drilling operation or by water testing.

Three-Dimensional Open Planar Separations (3-D)

The 3-D category indicates the presence of two or more intersecting planar discontinuities or open planes of separation that pass through the rock mass at the point of examination. The planar separation may form patterns or may be random in occurrence. Internal planar separations (IPS) terminate within the rock mass; mass planar separations (MPS) pass entirely through the rock mass and are infinite in extent in terms of design.

By geometric definition, three dimensions form a shape. This shape is often referred to as a joint block which has an average size and weight that can be estimated. The degree of interlock between joint blocks can be used to estimate the strength-of-foundation or the stability-of-excavation factor.

In the case of MPS, the attitude of the planes with respect to the slope or excavation is the chief design
factor. The ability of the planes to transmit water is estimated or measured as in the 2-D category above.

**Unit Weight**

Density or unit weight has been found to be one of the most useful and reliable means of communicating rock quality to design engineers and contractors, due to their past experience with rock. The unit weight is determined in the field with the aid of a spring-loaded "fish" scale and a bucket of water. A suitable sample is fastened to the scale with a short piece of string. The weight of the sample is determined first in air and then submerged in water. The weight of the string (wet and dry) is ignored. The unit weight is calculated by the following equation:

\[
\text{Unit Weight} = \frac{W_a}{(W_a - W_w)} D_w
\]

where \(W_a\) is the weight of the sample in air, \(W_w\) is the weight of the sample in water, and \(D_w\) is the density of water (62.4 pounds per cubic foot, pcf). The weight of the sample in air and water can be measured in either pounds or grams since the mathematical operation with the weights produces a dimensionless number.

The five categories of unit weight are: 1) greater than 160 pcf (2.55 g/cc) designated by A, 2) 150 to 160 pcf (2.40 to 2.55 g/cc) designated by B, 3) 140 to 150 pcf (2.25 to 2.40 g/cc) designated by C, 4) 130 to 140 pcf (2.10 to 2.25 g/cc) designated by D, and 5) less than 130 pcf (2.10 g/cc) designated by E. These five unit weight conditions are listed in the bottom line of Figure 1.

The unit weight design evaluation establishes the driving force in problems of slope stability, the relative usefulness of the rock material as a surface course or concrete aggregate, of the weight-volume relationship for estimates of haul cost, and unit weight establishes the degree of change due to change of weathering state.

As a general rule, the author has found that rock material having a unit weight greater than 160 pcf (2.55 g/cc) is suitable more than 50 percent of the time for use as road aggregate, concrete aggregate, riprap, or jettystone without laboratory testing. Rock material having a unit weight of 150 to 160 pcf (2.40 to 2.55 g/cc) may be acceptable for these uses but requires laboratory testing for confirmation. Rock having a unit weight less than 150 pcf (2.40 g/cc) is usually not acceptable for these uses, typically does not produce free-draining fill, and will probably degrade. Rock having a unit weight less than 130 pcf (2.10 g/cc) can usually be excavated by machinery but will likely degrade during excavation under abrasion of excavation equipment.

**Symbols and Notations**

The URCS employs a simple system of notation for graphic representation on geotechnical inventories, boring logs, maps, and sections. The notation registers rapidly in the mind and minimizes the required drafting effort. The letter design symbols for the four basic elements described above and shown in Figure 1 are grouped together in a four-letter notation.

The four letters are upper case and are underlined. The letter "A" denotes that the least design evaluation is generally required, while the letter "E" denotes the greatest design evaluation. The letter "O" in sequence indicates that no determination of that basic element was made; a lower case "O" after an upper case letter indicates that the value of the basic element was estimated.

The field notation should include both the letter symbol and the abbreviated basis written under the letter. For example:

Symbol on log: BCAD
Field notebook: B VFS C DQ A SRB 130
Explanation: B—Visually fresh state
C—Dent quality
A—Solid random breakage
D—Unit weight of 130 pcf

Design values can then be established for this material for the intended purpose.

Some combinations of states or qualities do not occur. For example, the CDS weathering state could not occur with RQ strength or 160 pcf (2.55 g/cc) unit weight. Neither could MQ strength material occur with MFS weathering.

Design values of the four basic elements are not equivalent even though the same letter notation might apply. In general, rock material designated AAAA will require the least design evaluation while EEEE will require the most.

**DISCUSSION**

Information pertaining to rock material or rock masses in current contract specifications or design memoranda is sketchy and ambiguous, to say the least, even when supported by laboratory test data. The terminology used in drilling logs and geologic
## BULLETIN OF THE ASSOCIATION OF ENGINEERING GEOLOGISTS

**PROJECT:** LOOKOUT CR. RSI  
**DISTRICT:** BLUE RIVER  
**DATE:** 5-2-80 TO 5-3-80  
**DRILL RIG:** CD-20 ACKER MK IV  
**HOLE DIA.:** 3 INCH  
**HOLE LOC.:** N1385 E460  

### WILLIAMETTE N.F  (EXAMPLE)  HOLE NO. DH-I

<table>
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<tr>
<th>ELEVATIONS</th>
<th>ROCK SOURCE</th>
<th>ROCK UNIT</th>
<th>MATERIALS</th>
<th>CORE RCY %</th>
<th>SPECIFIC DATA</th>
<th>COMMENTS</th>
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<td>2300</td>
<td>D.A.W. 5-3-80</td>
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<td>ROCK FRAGMENTS with SILTY SAND SOIL UNIT A Brown, moist, above plastic limit, stiff</td>
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<td>100%</td>
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</tbody>
</table>

**BOTTOM**  
DEPTH 55.0 FT.  
ELEV. 2245

**LOGGED**  
D.A.W. 5-3-80

---

Figure 3. Example of boring log utilizing URCS notation.
sections usually fail to provide understandable information to the contractor for purposes of bid estimates.

Here is an example of a rock description found in a typical contract specification or design memorandum:

"Slightly weathered, moderately hard, highly fractured, lightweight rhyolitic rock."

This information is sincerely intended to portray actual conditions existing at a site and will provide the basis of the design, the cost estimate, and the judgment of the construction method required as well as the basis for defending the owner from future construction claims or extras.

Descriptive terms such as these vary widely in meaning, depending on both individual and professional experience, and cannot be quantified with any degree of precision or uniformity. Consequently, design decisions, cost estimates, or construction methods based on such information also vary widely.

The URCS offers a suitable alternative to this ambiguous, descriptive approach. The term "unified" refers to the necessary unification of geology and engineering for geotechnical purposes. The URCS equivalent of the typical rock description for contract specifications and design memoranda is CCED. This simple notation is based on uniform acceptable procedures used to define design conditions.

The CCED notation indicates that the degree of weathering of the rock is the stained state (STS) or not representative of the standard design condition that exists at the site and that comparative data will have to be determined. The strength of the rock material is dent quality (DQ) and has a range of unconfined compressive strength of 3,000 to 8,000 psi (21 to 55 MPa), which is comparable to the
strength of concrete. The rock mass has three-di-

mensional planar separations (3-D), which will be

the primary design and construction consideration

with respect to stability, excavation, and material

use. The size, shape, volume and weight of the unit

joint block have not been defined and will have to

be determined as well as the continuity and attitude

of the planes and degree of interlock of the joint

blocks. Water transmission will have to be esti-

mated or measured. The unit weight of the rock

material is 130 to 140 pcf (2.10 to 2.25 g/cc), which

indicates that there will be full loads for hauling

equipment but that the material is probably not free

draining nor can it be used in load-bearing fills or

for surfacing.

This simple but well-defined verifiable design no-
tation is suitable for graphic abstracts, boring logs,

plans and sections, and other documentation. Since

it is based on basic design elements, the notation

provides a reliable means for decision. The notation

registers rapidly in the mind during scanning and

allows rapid comparison with several rock condi-
tions. Similarities and differences can be established

immediately. The simple notation minimizes draft-
ing effort. The notation prevents subjective con-
notation and allows recording of the significant in-
formation on a scale appropriate to the investigation.
The information can be checked and verified.

Examples of the notation in actual use on a boring

log and on a section are presented as Figures 3 and

4, respectively.

CONCLUSION

The URCS furnishes a means by which persons

from professional and technical disciplines who have

different experience levels can communicate in a

reliable and unambiguous manner about rock con-

ditions.

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APPENDIX 3.5
3.5 The Field-Developed Cross-Section

This appendix describes the field-developed cross-section. Reprinted by permission of the Association of Engineering Geologists, from the *Proceedings of the 39th Annual Meeting*, 1991, pp. 719 to 738.
THE FIELD-DEVELOPED CROSS-SECTION:
A SYSTEMATIC METHOD OF PORTRAYING DIMENSIONAL
SUBSURFACE INFORMATION AND MODELING FOR
GEOTECHNICAL INTERPRETATION AND ANALYSIS

Douglas A. Williamson, Forest Engineering Geologist (Retired),
USDA Forest Service, Eugene, OR 97402
Kenneth G. Neal, Senior Engineering Geologist,
GeoEngineers, Inc., Redmond, WA 98052
Dennis A. Larson, Assistant Forest Engineer,
USDA Forest Service, Olympic National Forest, Olympia, WA 98507

BACKGROUND

The field-developed cross-section, also referred to in the past as the ground-measured cross-section, was developed by Douglas A. Williamson during the late 1950s when the initial foundation investigation for Hills Creek Dam in western Oregon was being conducted. The need for a new system of portrayal was identified by Williamson when he found that standard geologic field methods did not lend themselves to prediction of subsurface conditions to be encountered by drilling. The system was refined during the 1960s during the Cougar Dam, Oregon, road location project, and during the years since. Williamson combined engineering survey measurement and notation methods with geologic subsurface interpretive methods to develop reliable systematic, reproducible, graphic, and dimensional subsurface portrayals.

The field-developed cross-section is applicable to nearly all types of site-specific engineering geologic investigations. It can be applied to excavation and placement of materials; foundations and slopes; mining engineering; specific development of groundwater, aggregate, mineral, and energy resources; storage and disposal sites; and for accurate graphic portrayal and analysis of significant features related to slope stability, seismicity, drainage, or other characteristics. The basic survey and portrayal standards described in this document can be applied to any of these applications. Variables include scale and minimum descriptive inventory data.

When various types of analytical sections are compared, the applicability of each can also be evaluated. A cross-section drawn from a 1:24,000 scale topographic base map provides generalized information only, because it misses even significant slope breaks between contour lines. A section developed from a plane-table map is superior because of scale, but it also lacks portrayal accuracy because contour lines are interpreted elevations except at survey stations. Surveyed cross-sections are far superior to those taken from contour maps because breaks in slope are based on actual measurements along the bearing of the section. Although they can be very accurate, most surveys are either plotted in the office by computer or manually. Lines between survey stations are assumed to be straight (intermediate topographic breaks are ignored). The computer plot is limited, and actual end-point relations may be lost.
Although it lacks the precision of higher-order engineering surveys, the field-developed cross-section (Figure 1), as the name suggests, is a field-developed portrayal of actual surface relationships and interpreted subsurface relationships. It is by far the most useful of all for observation and analysis of actual site conditions. It is not recommended as an alternative to engineering site surveys, but rather as a complement or supplement that provides rational data in an easily understandable form for decision making and design.

Figure 1. Geotechnical cross-section 30 for the Binder Timber Sale. (Blair and Scheible, 1983)

FUNDAMENTALS OF THE FIELD-DEVELOPED CROSS-SECTION

Elements of the Process

The field-developed cross-section is a geotechnical team effort in applying the scientific method to interpret and analyze site conditions for various engineering and resource applications. Each field-developed cross-section is a visual portrayal that provides an interpretation of site-specific subsurface conditions. The section can be used to explain each specific condition portrayed in terms understandable to the viewer. Information portrayed can be divided into component parts to define the problem or condition and can be used to explain complex parts of a feature and their relationships for geotechnical modeling and analysis. Portrayals are set up to facilitate organizational interaction so
individuals can discuss cross-sectional relationships based on their experience, with the net result being a discussion based on the shared experiences within the group.

As each field-developed cross-section is measured, data are gathered independent of scale. After data are reduced, the section is plotted at a scale appropriate for portrayal of conditions and application to the problem identified. This allows for considerable flexibility in selecting a scale appropriate for the identified site conditions.

The system provides for continuity over time. By following the system, cross-sectional relationship portrayals can be updated to reflect changes resulting from natural processes, such as ongoing slope movement, or site modification or development, such as excavation or placement of materials in a quarry. The system is designed for objectivity, producing verifiable and reproducible results.

Skills and Knowledge Required

An individual measuring a field-developed cross-section must have a strong background in geologic origin and process and the ability to interpret sequence of deposition, deformation, and landform development. The individual must be able to relate surface conditions to subsurface relationships in soil and rock. He or she must have a working knowledge of the Unified Soil Classification System (American Society for Testing and Materials, 1987) and the Unified Rock Classification System (Williamson, 1987), and be able to apply these systems in the field. The individual must be able to relate physical characteristics of materials to landforms and processes, and physical materials characteristics and distribution to subsurface water flow.

A basic knowledge of survey methods using basic tools (cloth tape, hand clinometer, and Brunton compass) is required in order to measure a field-developed cross-section. Proficiency at notetaking, basic skills in using hand-held calculators for reducing slope measurements to horizontal and vertical distances, and sufficient drafting skills to plot the cross-section in the field are also required.

Knowledge of design principles and construction methods is necessary to be proficient in selecting investigative standards. The individual must be capable of relating conditions and features encountered to foundations, excavations, slopes, use of materials, and permeability (FESUP). A working knowledge of slope stability and foundation analysis, the ability to relate strengths of materials to field tests, and a working knowledge of drilling and other subsurface exploration techniques used to confirm relationships and obtain test data related to soil, rock, and subsurface water are also required.

Applications of the System

The field-developed cross-section has been successfully used on numerous U.S. Army Corps of Engineer dam projects in Oregon. In addition to dams and powerhouses, the system has been applied to road cuts and fills, bridges, retaining structures, buildings, subdivisions, pits and quarries, waste sites, drainfields, sewer lines, water wells, water towers, water transmission lines, trails, landings for logging, slope stability analysis, channel and shorelines stabilization projects, fish ladders, prediction of impacts from reservoir drawdown, ski lifts, sanitary landfills, fire suppression dip tanks for helicopters, airport landing strips, pavement design, power transmission line foundations, drainage
structure design, blast design, and rock bolt design. The system has been applied during all project phases, from planning through construction, including interpretation and documentation of as-built conditions.

Field-Developed Cross-Section—Step-by-Step Procedure (Serial Order of Work)

The following steps outline the process, in serial order, of completing a site investigation using the field-developed cross-section investigative method:

1. **Develop general knowledge of the area.** This may be through examination of state maps and reports, theses, minerals or oil/gas maps, previous geotechnical investigations or explorations; discussions with those familiar with the site; and by developing familiarity while traveling to the site.

2. **Establish/select survey control.** Survey control is expediently selected. Base elevation may be estimated from map contours or from an existing site survey.

3. **Identify landmarks related to previous investigation or exploration.** Data tied to these landmarks will be used as cross-sectional relationships are developed.

4. **Examine site-specific relationships.** Landform and process are interpreted and related to the distribution of soil and rock materials, and water. Soil and rock units may be preliminarily established at this time.

5. **Select typical sections.** The number and locations of sections selected are tied to the type(s) of feature(s) or process(es) being investigated, the complexity of the site, and the proposed applications of the data.

6. **Measure each section.** Sections are typically measured using cloth tape, a hand clinometer, and a Brunton compass. Measurements include all slope breaks. The significance of each slope break (that is, survey station) is described in the field notebook. Since slope breaks are most commonly a result of changes in materials strength characteristics, many times they are contacts between soil and rock units. Measurements of contact orientation (strike, dip, and surface trace) are normally denoted where appropriate in comments for the station in the field notebook.

7. **Determine the number and sequence of soil and rock units.** Soil and rock units are designated locally, based on stratigraphic relationships and engineering strength characteristics. Soil units are designated Soil Unit (SU)-A, SU-B, SU-C, etc., from bottom to top of the sequence. Rock units are designated Rock Unit (RU)-10, RU-11, RU-12, etc., from bottom to top of the sequence.

8. **Observe, gather, test, and document minimum engineering information for each soil and rock unit.**

9. **Calculate and tabulate.** Each station established must have a survey station and elevation.

10. **Draw relationships in cross-section and plan.** This step involves plotting survey stations; drawing surficial relationships and conditions; and drawing interpretations of subsurface conditions, including distributions of SUs and RUs, estimated location(s) of water, original ground lines as related to existing excavated and filled landforms and slope movements, and locations and continuities of "failure planes" and other mass structural surfaces.

11. **Consider design alternatives.** Relate design alternatives to site conditions/relationships.

12. **Consider the need and feasibility of drilling or other subsurface exploration.** If appropriate, plan and apply exploration to confirm subsurface interpretations and to gather test data. Apply confirmational information to each cross-section as needed.

Appendix 3.5
13. **Reduce data for each SU and RU to strength values.** Select cohesion and angle of internal friction (C and $\phi$) or other value as appropriate. Zone site on the basis of physical strength characteristics as applicable to the type of project to be analyzed.

14. **Apply slope stability analysis, bearing capacity formulae, permeability formulae, blasting formulae, or other formulae as needed.**

15. **Apply analyses to intended use.** Design a solution appropriate for the site characteristics to be encountered.

### DETAILS OF CROSS-SECTIONAL PORTRAYAL

**Items Portrayed**

The field-developed cross-section is designed to portray 1) topography; 2) rock line; 3) soil and rock units (SU and RU); 4) mass and other significant planar separations; 4) subsurface water-bearing zones; 5) springs (should be dated and initialed, and volume estimated to identify seasonal characteristics); 6) surface water (see item 6); 7) original ground line (where topography has been altered by development or construction); 8) constructed features such as road shoulders, base and surfacing, road numbers, drainage facilities, embankments, structures; 9) minimum engineering information, which includes descriptions and engineering classifications of materials; 10) existing survey data; 11) intersections with other field-developed cross-sections; and 12) other resource data (mineral, aggregate, energy, etc.) where appropriate.

The location measured in section must be selected so that 1) it portrays conditions typical of what it represents; 2) it is normal (at right angle to) or parallel to the slope; and 3) it extends beyond the limits of the feature portrayed and shows adjacent related characteristics. The location should tie (if possible) to highly visible end points such as trees, snags, rock points, or constructed features.

Field-developed cross-sections are located in a configuration to provide all pertinent information required for the application specified. For a slope movement feature, the minimum configuration may simply be a single section following the axis of movement. For a rock source, the minimum configuration is two perpendicular sections that display the distribution of soil and rock units in three dimensions, and that provide dimensional relationships for crude volume calculations. For ground water, the simplest configuration is a cross-valley section designed to display conditions below the valley floor. Figure 2 depicts the minimum configuration required for foundation investigations for various types of structures. The configurations shown provide three-dimensional portrayal under each footing. For all of the applications discussed, the minimum configurations given are for relatively simple sites with few variations in slope geometry or lateral discontinuities in soil, rock, or drainage characteristics. Additional sections, both normal to and across the slope, may be necessary for more complex or large sites to adequately portray the differing conditions.

Roadway projects require a combination of different configurations, depending on site characteristics and design requirements. The surveyed line (or existing roadway in the case of a reconstruction project) is subdivided into segments having similar topographic characteristics, materials distribution, and drainage characteristics. Where slopes are smooth and uniform, without complex or adverse relationships requiring dimensional portrayal, typical sections are sketched. Where slopes are irregular or conditions are complex, typical sections are measured at right angles to the slope, and if possible,
Figure 8. Portrayal of drill holes on field-developed cross-sections.

REFERENCES


consistency, fabric, or water content. Virtually every natural slope break encountered will fall into one of the categories previously listed. Stations should also be established at significant mass planar features not affecting slope symmetry, and at locations to denote significant features of constructed facilities.

Each station is marked with a bright-colored ribbon and labeled with a waterproofing marker, indicating cross-section and traverse point number (A10, for example). It is preferable to start numbering at 10 or 20 to allow for later extension of the section if necessary. After the survey is completed and data are reduced, stations should be relabeled to indicate station designation on the traverse (10+94, for example).

Survey Notetaking

Survey notes are taken in a weather-resistant level book using the headings shown in Figure 3, which shows the standard notetaking format. Using the standard format, survey notes are taken independently of soil, rock, and water inventory data (Figure 4). Minimum descriptive information for each traverse point is listed in Table 1.

Measurement of the Section

The azimuth of the field-developed cross-section is measured with a Brunton compass. Distances and slope angles are measured using cloth tape and a hand clinometer to an accuracy of 0.1 feet and 0.5 degree, respectively. Measurements can be taken either of slope distance and angle, or horizontal distance and rod. Distances are measured at eye height (eye of person running the section to his or her eye height on the field partner) when measuring slope distance and angle, or from eye height to the same elevation (along a level line) at the station when measuring horizontal distance and rod (Figure 5). The section is measured from bottom to top of slope because running the section uphill is best visually.

Two-Person Method - The two-person method is preferred because it (a) allows for the greatest amount of ground contact and observation time per section, and (b) allows for the greatest flexibility in measurement (both along and adjacent to the section). In the two-person method, the person taking notes establishes the starting point (if not done earlier) by letter designation and number, measures the orientation of the section using a Brunton compass, and labels the first traverse point. The person not taking notes (rodman) moves ahead on line to the next station. Measurements are taken station to station and the team moves upslope in tandem. The upslope person tags and labels stations as the downslope person gathers the geotechnical inventory data and sketches subsurface interpretations in the field book.

One-Person Method - The one-person method is used (a) when a second team member is not available, or (b) when terrain characteristics make the two-person method difficult or impossible. The one-person method requires establishing an eye level station upslope within line-of-site distance of the bottom station. This can be done by measuring to eye height on a second team member (difficult terrain circumstances) in a stable, stationary position far upslope on line, or by establishing a station upslope at a tree or other stationary object by driving an aluminum nail at eye height and marking and attaching the end of the tape to the nail (Figure 5). The person measuring the section occupies the farthest station, measures its position relative to the stationary person or reference...
Some complex sections can most easily be measured using a combination of these methods. Point, labels the station, gathers all inventory data, and then moves upslope station by station until each point is described and located relative to the reference point.
Closure of Sections

The field-developed cross-section requires closure similar to formal surveys. Closure may be accomplished by 1) measuring between end points or intermediate points of parallel sections, or 2) measuring between end points of the same section (to ensure that the sum of the parts equals the whole).

Figure 4. Standard soil, rock, and water note format for field-developed cross-section.
### Table 1: Field Identification and Application

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Field Identification</th>
<th>Application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>Visual inspection</td>
<td>Blueprints</td>
</tr>
<tr>
<td>Sand</td>
<td>Texture analysis</td>
<td>Plans</td>
</tr>
<tr>
<td>Silt</td>
<td>Tactile test</td>
<td>Specifications</td>
</tr>
<tr>
<td>gravel</td>
<td>Color test</td>
<td>Reports</td>
</tr>
<tr>
<td>organic</td>
<td>Odor test</td>
<td>Certificates</td>
</tr>
<tr>
<td>mixed</td>
<td>Acoustical analysis</td>
<td>Licenses</td>
</tr>
<tr>
<td>loam</td>
<td>Physical analysis</td>
<td>Permits</td>
</tr>
</tbody>
</table>

**Unified Soil Classification**

**Unified Rock and Soil Classification Systems**
TABLE 1 (CONTINUED)
UNIFIED ROCK CLASSIFICATION (URCS)
ROCK MASS STRENGTH ESTIMATES - BASIC ELEMENTS

<table>
<thead>
<tr>
<th>EFFECT OF WEATHERING</th>
</tr>
</thead>
<tbody>
<tr>
<td>WEATHERED</td>
</tr>
<tr>
<td>SAND SIZE</td>
</tr>
<tr>
<td>COMPLETELY DECOMPOSED STATE (ODR)</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>PLASTIC</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MINERAL GRAIN BONDING</th>
</tr>
</thead>
<tbody>
<tr>
<td>REMOLDING</td>
</tr>
<tr>
<td>MOLDABLE (FRIABLE) (FBL)</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>&lt;1000 PSI (7 MPa)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TRANSMITS WATER</th>
</tr>
</thead>
<tbody>
<tr>
<td>YES</td>
</tr>
<tr>
<td>3 DIMENSIONAL PLANE OF SEPARATION (3D)</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>LATENT</td>
</tr>
<tr>
<td>PLANE OF BREAKAGE</td>
</tr>
<tr>
<td>LESS THAN</td>
</tr>
<tr>
<td>130 LBS/CU FT</td>
</tr>
<tr>
<td>(2.80 Mg PER CU M)</td>
</tr>
<tr>
<td>(&lt;130)</td>
</tr>
<tr>
<td>E</td>
</tr>
<tr>
<td>UNIT WEIGHT</td>
</tr>
<tr>
<td>LESS THAN</td>
</tr>
<tr>
<td>130 TO 140</td>
</tr>
<tr>
<td>LBS/CU FT</td>
</tr>
<tr>
<td>(2.10 TO 2.55</td>
</tr>
<tr>
<td>Mg PER CU M)</td>
</tr>
<tr>
<td>(330)</td>
</tr>
<tr>
<td>GREATEER THAN</td>
</tr>
<tr>
<td>160 LBS/CU FT</td>
</tr>
<tr>
<td>(2.55 Mg PER CU M)</td>
</tr>
<tr>
<td>(&gt;160)</td>
</tr>
</tbody>
</table>

The purpose of closure is to give the engineering geologist measuring the section a means of checking his or her work, to ensure that all points have been measured within a reasonable range of accuracy. While not intended to represent a legal survey, a lack of careful measurement can mislocate a key reference point for foundation location or not provide a proper topographic profile for slope stability analysis. Error in closure should not generally exceed 1 foot per 100 feet of line. Closure error should be noted on the rough draft section and adjustments made during preparation of final drawings.

Data Reduction

Data can be reduced either in the field notebook (best for field reduction) (Figure 6a) or on a separate piece of paper (most accurate method if reduction is done in office or if section is to be plotted by someone other than the person who measured it) (Figure 6b). Data reduction is accomplished by using a hand-held calculator to compute vertical and horizontal distances from slope distance and vertical angle in degrees. This can be done by converting from polar to x-y coordinates or by using trigonometric functions.
Figure 5. Measuring distances between traverse points.
Once horizontal and vertical distances are calculated for all points, then stations and elevations are established. The stationing usually starts with 10+00 to allow extension at a later date if necessary. Where the section crosses a survey line, such as a P-line, the section is referenced to the station crossed. Where the section follows an existing survey line (such as bridge centerline), the existing stationing is used. Stationing for each traverse is calculated at a point with a number of elevations.
Figure 6b. Reduced field notes on separate reduction form.

Point is then calculated based on horizontal distance from 10+00. Stations are assigned such that station numbers increase from left to right as viewed looking ahead on line along the referenced survey, if one has been established. Elevations are calculated and listed for each station on a reduction sheet or in the field notebook.
### Table 2 - Application of Various Cross-Section Scales

<table>
<thead>
<tr>
<th>TYPE OF PROJECT</th>
<th>INVENTORY</th>
<th>ANALYSIS</th>
<th>REPORT</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SCALE</td>
<td>METRIC</td>
<td>METRIC</td>
</tr>
<tr>
<td>SLOPE STABILITY</td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td></td>
<td>1:300</td>
<td>1:300</td>
<td>1:600</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 3 m.)</td>
<td>(1 in. = 25 ft.)</td>
<td>(1 cm. = 6 m.)</td>
</tr>
<tr>
<td>STRUCTURAL FOUNDATION</td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td>CUTSLOPE EXCAVATION</td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td>ROAD BASE AND SURFACING</td>
<td>1:50</td>
<td>1:60</td>
<td>1:50</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 0.5 m.)</td>
<td>(1 in. = 5 ft.)</td>
<td>(1 cm. = 0.5 m.)</td>
</tr>
<tr>
<td></td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td>EMBANKMENTS</td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td>DRAINAGE</td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td>ROCK AND AGGREGATE SOURCES</td>
<td>1:100</td>
<td>1:120</td>
<td>1:100</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 1 m.)</td>
<td>(1 in. = 10 ft.)</td>
<td>(1 cm. = 1 m.)</td>
</tr>
<tr>
<td></td>
<td>1:600</td>
<td>1:600</td>
<td>1:600</td>
</tr>
<tr>
<td></td>
<td>(1 cm. = 6 m.)</td>
<td>(1 in. = 50 ft.)</td>
<td>(1 cm. = 6 m.)</td>
</tr>
</tbody>
</table>

### Selection of Scale

Table 2 shows the range of scales generally appropriate for various field-developed cross-section applications. All sections in a report should be portrayed at the same scale, if possible, for ease of understanding and application.

### Section Layout

The field-developed cross-section may be plotted on a variety of materials. For field plotting, particularly in areas where conditions are commonly wet, layout on frosted mylar has been the most successful. Commercially prepared sheets 8-1/2" × 11", 11" × 17", or 22" × 36" with a 0.1" and 1" engineering grid are commonly used for field plotting.

Layout is completed by first establishing horizontal and vertical reference elevations and stations at the selected scale, plotted top and bottom and both sides. Stations are plotted from left to right unless there is a special need (such as to have field-developed cross-section stationing easily discernible from a yet unestablished site survey).
cross-sections too long to plot on one sheet, end points of sections and drawing reference points are matched carefully for taping. Each facet must be labeled carefully to ensure a later match for final drafting.

Each field-developed cross-section must have a bar scale. The title block for each section is located in the lower right-hand corner for uniform referencing. The title block must include project, location, and type of survey used. Bearings and bends in the section are shown along the top of the page (Figure 7).

**Portrayal of Subsurface Conditions**

The portrayal of subsurface conditions is the most critical aspect of this method because 1) it is the primary reason for measuring the section, and 2) it is the major element that distinguishes the field-developed cross-section from all other techniques. The portrayal of subsurface conditions requires a correct geological site interpretation (geomorphology, stratigraphy, and structure) to be accurate.

The rock line is the top of recognizable rock in place observed as a rock unit. Rock in this context may range from completely decomposed to solid. The rock line is portrayed based on rock exposures and projection of rock surfaces, and the interpretation of surficial features and materials strengths and weaknesses. Rock and soil units are shown by surficial relationships along the section and by projection from adjacent outcrops. Contacts between soil and rock units, and mass and other significant planar separations are drawn to portray their apparent dip (in the plane of the section). When intersecting sections are drawn, the same plane must be portrayed at the same elevation at the intersection on both sections.

Drilling may be completed at a later date to confirm subsurface interpretations. Drill hole location requires remeasurement from an established station on the section. When drilled on the section, drill holes are drawn to scale on the section (including any surface modification for drill set up and operation). Holes drilled away from the section may be portrayed in one of three manners, depending on the objective of the portrayal (Figure 8). The most common method of portrayal is to project the hole into the plane of the section at right angles to the section. Hole locations are designated as ____ ft. right or left of section (DH-1 in Figure 8). Alternative and less common methods include 1) projecting to the section at the same elevation as the top of the hole; hole locations are designated as ____ ft. at ____ (bearing) from the station shown (DH-2 in Figure 8); and 2) projecting along strike of bedrock to the intersecting point on the section; hole locations are designated the same as above (DH-3 in Figure 8).

In all instances, drill holes are drawn to portray true top of hole elevation (that is, the top of hole may lie above or below the elevation of the corresponding point on the section).

**ANALYSIS AND APPLICATION**

Analysis of data gathered during measurement and confirmation of field-developed cross-sections for various purposes includes the following steps and phases.

1. **Data Reduction:** During this phase, raw field descriptions and classifications, and laboratory and drill test data are evaluated and sorted to determine pertinence and
applicability to the problem or need. Extraneous data are discarded. Information to be applied is refined from description and classification into numerical values such as bearing capacity, unconfined compressive strength, cohesion, angle of internal friction, mass or weight, confinement, permeability, porosity, rate of weathering, rate of water transmittal, or other more specialized values.

2. **Zonation:** Strength or other values pertaining to the problem to be solved are displayed on an analysis copy of the field-developed cross-section. Differences and similarities in values are noted. Boundaries reflecting changes in value are drawn. A zone diagram is superimposed or overlaid on the field-developed cross-section (Figure 7), reflecting topographic characteristics, zone boundaries, and pertinent numerical values. The zone diagram is applied to design and contract applications. A copy of the analysis section is kept for reference.

3. **Design Analysis of Alternatives:** Alternative lines and grades are plotted on copies of the zone diagram. Design suitability and limitations, construction problems, probable impacts from construction, and material needs are estimated either by judgment or through use of a mathematical model, if a suitable model is available or can be developed.

**SUMMARY AND CONCLUSION**

The field-developed cross-section method was originally developed to provide a systematic and reproducible method of collecting data for portrayal of subsurface conditions expected to be encountered during drilling. It has evolved into a system that has significantly improved the efficiency and reliability of the investigative process for 32 (to date) different kinds of engineering geological project applications.

The field-developed cross-section investigative method requires systematic measurement, observation, testing, classification, and recording of data at each station. Hence, the process yields reproducible results, largely independent of the personal skills and knowledge extending beyond required basic field tests, plus skills in descriptive geometry, and geomorphic and structural interpretation. A journeyman engineering geologist can, with a minor investment in training, direct field crews of less experienced geologists to obtain very reliable and accurate field data, and then can apply his or her experience and knowledge to analysis and application of that data. Thus the system allows the journeyman engineering geologist to extend his or her skills through utilization of others.

When conditions portrayed are tied together and projected in plan as well as section, three-dimensional relationships are readily visible for the type of analysis needed. Potential alternatives can be applied visually and dimensionally directly to the field-developed cross-sections. Often visual observation of relationships applied to the alternative will yield answers directly, without necessarily requiring the application of statistical or other mathematical analysis methods. If mathematical analysis is required, the field-developed cross-section provides a reliable model.

Since selection of stations is related to real ground conditions, the data, once tabulated, is independent of scale. The scale selected for the section is one that best portrays the conditions investigated. The portrayal of conditions allows the resulting field-developed cross-section and zone diagrams to be used as a medium of communication. This system readily bridges the barriers of discipline and scientific nomenclature.
Figure 7. Field-developed cross-section and foundation zone diagram.
Figure 7. (Continued)
perpendicular to the survey line. Each section must extend beyond the feature requiring portrayal, at least 100 feet either side of the surveyed line, or to a ridge top or valley bottom if closer than 100 feet. The section must show the range of conditions typical for that segment. For road segments involving stream crossings, where special structures may be designed or where the alignment crosses a slope stability feature, the field-developed cross configuration for that specific type of investigation is used.

Selection of Stations

Once end points of the field-developed cross-section are selected and marked, stations to be surveyed can be established. Stations can be selected either before or while measuring the section. The line should be brushed if necessary before selecting stations. (NOTE: Be sure the landowner gives permission to brush.) Points selected for survey stations are usually slope breaks because slope breaks indicate a change in materials strength characteristics, such as (in rock) a change in texture, hardness, fabric, weathering, or structure; or (in soil) a change in texture, plasticity, compactness or
APPENDIX 3.6
3.6 Geotechnical Exploration—Drive Probe Method

Douglas A. Williamson, Geotechnical Engineering Geologist, USDA Forest Service (Retired)

Introduction

The relative density probe is a simple exploratory device used to determine the distribution and estimated strength of the subsurface soil units and decomposed rock units. The drive probe can also be used to determine the presence of water and subsurface water levels. It requires some equipment and moderate work effort, but it does define strength parameters of the soil units. Once set up, it is much quicker than hand augering but does not provide a sample to determine plasticity. One or more auger holes can be used to classify the subsurface materials, and the drive probe can be used to define the subsurface for the rest of the project area.

The equipment is relatively inexpensive (less than $500) and can be either specified and ordered or fabricated in-house. The drive probe is most effective when used in conjunction with conventional geotechnical soil exploration to inexpensively extend the known conditions revealed by conventional drilling. The drive probe can also be used alone and has been found to be accurate and discrete in defining subsurface conditions for difficult access projects. It has been proven through geotechnical work over a period of more than 4 years.

“Relative density” in this document is defined as the estimated strength of subsurface materials determined by the resistance to penetration. The resistance to penetration is measured in blows-per-foot of an 11-pound circular hammer, freely falling “39 inches,” striking a coupling, and driving a 1-inch diameter solid end area. Note the “39 inches” is the distance between couplings on a 4-foot pipe length minus the length of the hammer.

Equipment

The equipment for the drive probe consists of segments of 1/2-inch galvanized pipe (3/4-inch outside diameter), cut into 4- and 5-foot lengths and threaded on each end. Commercially, 1/2-inch galvanized pipe comes in 21-foot lengths, so the pipe is cut into one 5-foot length and four 4-foot lengths to eliminate waste. Smooth-walled pipe couplings with full threads (1-inch outside diameter) are used to join the pipes together as the hole is advanced. The 5-foot length is used for the initial drive. The starting 5-foot length is randomly drilled with 3/16-inch diameter holes to allow water to enter the pipe as it is driven into the ground. The driving end of the pipe is a coupling with a pipe plug in it.

It is critical to the operation that the threads on the ends of the pipe are no longer than those which will “meet” in the center of the full-thread couplings when two pipes are joined. This allows the driving force to be transmitted through the pipe.
ends and not through the threads. The pipe lengths can be made up in advance with a coupling on one end, ready for adding to the drive string.

An 11-pound, circular, mild steel hammer is used. The total hammer length is 7-3/4 inches, including the guards. The driving section is 7 inches long (minus the guards) and 2-1/2 inches in diameter. The driving section has a hole in the center that is 7/8 inch in diameter through which the pipe will pass freely. Welded on each end of the driving section are round hand guards. The hand guards are 4 inches in diameter and made from 3/8-inch-thick mild steel plate.

Additional equipment needed includes: two pipe wrenches (preferably rigid brand) of the magnesium-aluminum variety for convenient weight; a plastic 5-gallon bucket to carry tools and equipment, to add water to the hole, and to stand on if necessary; an electric water level measuring device (m-scope); a funnel; a 5-foot measuring stick; yellow keel type marking pencil; a field notebook and pencils; and a folding shovel. Several spare couplings are also convenient.

A 1/2-inch pipe die and die stock, a pipe cutter, a pipe reamer, a pipe vise, and a 1/2-inch "easyout" for removal of broken pipe threads from couplings are convenient and economic in-house equipment. A supply of smooth-walled, full thread pipe couplings should be kept on hand.

A drive probe is designed for a one- or two-man operation for either technical or non-technical personnel. Depths of up to 30 feet have been achieved without unusual effort. When "refusal" is reached with "free-falling method," the hole can be continued by "hand-driving" methods to "absolute refusal."

A small portable tripod is useful if extensive drive probe drilling is done. The tripod can be made from three 10-foot lengths of 1-inch diameter electrical conduit joined at one end by a 1/2-inch diameter bolt. A convenient sheave suitable for use with a 3/4-inch diameter rope is attached to the bolt at the top of the tripod. The hammer can then be raised and lowered by attaching the rope to the hammer and pulling on the rope. A loop can be tied in the rope, and a foot can be used to raise and lower the hammer. This method is especially useful when "pulling a string." Gloves are a must when driving by hand or using a rope. See figure 3.6.1. for a schematic of a drive probe.

**Procedure**

The site selected for the probe is cleared of the surface duff or organic material to expose the upper surface of the soil deposit. The starting 5-foot length is measured and marked in 0.5-foot intervals with the keel or marking pencil. The 4-foot length with the hammer on it is screwed into the top of the 5-foot length. The hammer is then raised up to the coupling on the upper end of the 4-foot length and allowed to fall freely and strike the coupling on the upper end of the 5-foot length. A coupling should always be screwed into the top of the 4-foot length to prevent the hammer from flying off the top of the pipe when raising the hammer.

As each segment is driven, the number of blows required to advance the hole for each one-half foot (6 inches) is recorded. After driving 5 feet, the amount of open hole is measured and the hole checked for the presence of water. The 4-foot pipe segment is unscrewed and the hammer is moved to the uppermost pipe as each additional 4-foot segment is added to the "string." The bottom pipe segment is
marked each time. The hole is then advanced another 4 feet. The open hole and water level is again checked, and so on, until “refusal.”

The friction of the pipe in the hole is primarily between the couplings and the sides of the hole, rather than all of the pipe in the hole. This causes easier advance of the hole than if all of the pipe were in contact. It is recognized that the friction increases slightly with each additional coupling in the hole, but with this type of operation it is negligible. In granular soil materials, all of the pipe is probably in contact with the sides of the hole.

The pipe lengths can be removed from the hole for inspection of thread conditions, for cleaning and removal of soil material in the pipe, or for recovery of the pipe for further use. The hammer is driven “upward,” striking the coupling on the top of the uppermost pipe. Repeated blows remove whatever length is desired. It may be convenient to have a 2-foot segment for pipe removal for a more convenient driving length. Shorter people may have to stand on the bucket when starting the hole or when removing pipe from the hole. The pipes can either be retrieved or left in the hole for later water measurements.

Field Notebook

The headings for the information to be recorded in the field notebook are: project, date, hole location and elevation, and crew. Page headings include: depth, from, to, interval, blows, blows-per-foot, open hole, and depth to water. “Write in the rain” level books are recommended.

Calculations

When doing drive probe exploration, the blows needed to advance the hole for the 0.5-foot (6-inch) interval are doubled to get the “blows-per-foot” representing the strength of the material for that interval. Six inches are driven in order to be sure of detecting a layer 1-foot or more in thickness. The longer the interval driven, the less likely that “thin” layers will be detected. Any depth interval can be driven and converted to “blows-per-foot” for that interval. In all cases, whatever interval is advanced, the blows-per-interval are converted to blows-per-foot, the “standard of comparison” for that given material. The blows-per-foot have been found to be a discrete and reliable means of correlating between drive probe holes.

When a hole is drilled into a soil deposit by means of geotechnical driving methods, there is a “typical anticipated result.” This anticipated result is that the number of blows necessary to advance the hole will increase with depth to a point at which the hole cannot be advanced. In other words, soil deposits normally increase in strength and density with depth. When doing drive probe drilling, the result that is of most importance to a subsurface strength assessment is the detection of a relatively weaker layer revealed by a “blow-count-reversal.” A “blow-count-reversal” occurs when the blows-per-foot decrease with depth rather than increase. This condition indicates the strength and density of the soil material in that interval has decreased. It is this “blow-count-reversal” that is of prime significance in assessing foundation conditions or slope stability.

Water Measurement

Drive probe drilling allows static water levels to be measured and simple permeability tests to be done. When water levels, measured in the pipe during and after driving, appear inconsistent with previous measurements, water can be added to
the pipe to clean out the holes in the tip. The depth-to-water can be checked in the pipe after a period of time to see if there is a change in water level. Water can be added to the holes already in the ground, at any time, to check on the reliability of the previously measured static water levels.

There appears to be a “natural sealing” between the pipe and the plastic soil materials preventing water movement up along the pipe. Metered water flows can be added to the pipe to do maintained-head tests. Falling-head tests can be done by timing the drop in water level. From these data the permeability can be calculated.

Water can be directly added to the pipe with the 5-gallon bucket or by means of a back-pack tank. The hydraulic head of the water in the pipe will usually clear the holes in the tip. When adding water with a bucket, a “screw-on” type funnel is convenient but any funnel will do. If a back-pack tank is used, a 1/4-inch plastic tube attached to the tank is inserted in the pipe to the bottom of the pipe. The tube is then moved up and down to flush out any soil material filling the holes in the tip.

Log Form for Drive Probe Exploration

A sheet of 8-1/2- by 11-inch graph paper is used for a log form. The depth-in-feet is recorded along the left side of the sheet and the blows-per-foot are shown along the top. A convenient scale is used, usually two small squares equals 1 foot on a 10th grid. The blows-per-foot/interval are shown as a bar graph from left to right which indicates “relative strength” according to length. The usual interval is 0.5 foot (6 inches), although any interval can be shown. Elevations are also shown along the sides of the sheet. Interpretative lines are drawn showing correlations of hole segments having similar blows-per-foot. These correlations indicate changes in material and strength.
Figure 3.6.1—Schematic of a drive probe.
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