RECOMMENDED PROCEDURES FOR IMPLEMENTATION OF DMG SPECIAL PUBLICATION 117 GUIDELINES FOR ANALYZING AND MITIGATING LANDSLIDE HAZARDS IN CALIFORNIA

ASCE
American Society of Civil Engineers

Committee organized through the
ASCE Los Angeles Section Geotechnical Group
Document published by the
Southern California Earthquake Center

SCEC

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Implementation Committee:

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June 2002

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Document published by the
Southern California Earthquake Center

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The Southern California Earthquake Center (SCEC), headquartered at the University of Southern California, is a regionally focused organization founded in 1991 with a mission to gather new information about earthquakes in Southern California, integrate knowledge into a comprehensive and predictive understanding of earthquake phenomena, and communicate that understanding to end-users and the general public in order to increase earthquake awareness, reduce economic losses, and save lives. Funding for SCEC activities is provided by the National Science Foundation (NSF) and the U.S. Geological Survey (USGS). An outstanding community of scientists from over 40 institutions throughout the country participates in SCEC. The SCEC Communication, Education, and Outreach Program offers student research experiences, web-based education tools, classroom curricula, museum displays, public information brochures, online newsletters, and technical workshops and publications.

The cover photograph depicts a landslide that developed in the Ramona oilfield, north of San Martinez Grande Canyon, about 9 km east-northeast of Piru, California. The landslide is 600 m long, 100-150 m wide, and has an estimated volume of about 1 million cubic meters. During the Northridge earthquake (January 17, 1994), the landslide moved downslope about 15-25 meters. (Photograph courtesy of Randall Jibson, U.S. Geological Survey)

June 2002

ACKNOWLEDGMENTS

With the implementation of the Seismic Hazards Mapping Act in California, general guidelines for evaluating and mitigating seismic hazards in California were published by the California Department of Conservation, Division of Mines and Geology in 1997 as Special Publication 117. Building Officials in the Department of Building and Safety of the City of Los Angeles and the Department of Public Works of the County of Los Angeles requested assistance in the development of procedures to implement the requirements of the DMG SP 117 Guidelines and the Seismic Hazards Mapping Act for projects requiring their review. Cooperation was sought from other agencies in southern California and officials from the Counties of Riverside, San Bernardino, San Diego, Orange, and Ventura agreed to participate. In addition, the Division of Mines and Geology lent support to this effort.

The request to prepare implementation guidelines for the hazard analyses required in SP 117 was made through the Geotechnical Engineering Group of the Los Angeles Section of the American Society of Civil Engineers (ASCE) in the latter part of 1997. A group of practicing geotechnical engineers and engineering geologists was assembled to form a committee to develop implement procedures. It was decided to deal with liquefaction and landslide hazards separately. The liquefaction implementation committee completed its work in March 1999 with the publication of a set of guidelines by the Southern California Earthquake Center (SCEC) located at the University of Southern California in Los Angeles. The landslide hazard analysis implementation committee began its work in August 1998. The landslide hazard analysis implementation committee had the following members:

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The over 3-1/2 years effort of the committee members to study, evaluate, discuss, and formulate these guidelines is greatly appreciated. The summation of those consensus efforts is presented in this report.

The committee was organized by the southern California section of the Association of Civil Engineers and the City and County of Los Angeles Departments of Building and Safety and Public Works. The committee has, however, performed its work independent of those entities. The document represents the work of the committee. Although the document has been peer reviewed, the information and opinions presented are those of the committee and have not been endorsed by ASCE, SCEC, or the City or County of Los Angeles.

Appreciation is given to those who have taken their time to review this document and have provided many wise comments and suggestions: Professors Jonathan D. Bray and Raymond B. Seed of U.C. Berkeley, Professors Ellen M. Rathje and Stephen G. Wright of the University of Texas at Austin, Dr. Leland M. Kraft, Dr. Neven Matasovic, Dr. Edward Kavazanjian, Dr. Marshall Lew, Boris O. Korin, Allan E. Seward, and Larry K. Stark. Review comments were also made by John A. Barneich, S. Thomas Freeman, Yoshi Moriwaki, Sarkis V. Tatusian, and John T. Waggoner of GeoPentech and Robert A. Larson, County of Los Angeles.

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1 INTRODUCTION

1.1 OVERVIEW

Analysis of the static and seismic stability of natural and manmade slopes is a challenging geotechnical problem. Often, different professionals analyzing the same problem will estimate a wide variation in expected performance. That variation results from variable levels of care in site exploration, laboratory testing, and the performance of stability analyses. Proper static slope stability analysis requires an accurate characterization of:

1. Surface topography,
2. Subsurface stratigraphy,
3. Subsurface water levels and possible subsurface flow patterns,
4. Shear strength of materials through which the failure surface may pass, and
5. Unit weight of the materials overlying potential failure planes.

The stability calculations are then carried out using an appropriate analysis method for the potential failure surface being analyzed. A seismic slope stability analysis requires consideration of each of the above factors for static stability, as well as characterization of:

1. Design-basis earthquake ground motions at the site, and
2. Earthquake shaking effects on the strength and stress-deformation behavior of the soil, including pore pressure generation and rate effects (which can decrease or increase the shear strengths relative to the static case).

All of the above-enumerated factors are vital for proper analysis of static and seismic slope stability, although some are more easily characterized than others.

Two factors that are particularly challenging to characterize accurately are subsurface stratigraphy/geologic structure and soil shear strength. Subsurface characterization requires a thorough exploration program of borings, cone penetration tests, and/or trenches, and must identify the potentially critical soil zones. Characterization of representative soil shear strength parameters is an especially difficult step in slope stability analyses due in part to the heterogeneity and anisotropy of soil materials. Furthermore, the strength of a given soil is a function of strain rate, drainage conditions during shear, effective stresses acting on the soil prior to shear, the stress history of the soil, stress path, and any changes in water content and density that may occur over time. Due to the strong dependence of soil strength on these factors, methods of soil sampling and testing (which can potentially alter the above conditions for a tested sample relative to in-situ conditions) are of utmost importance for slope stability assessments.

This report provides guidelines on each of the above-enumerated factors, with particular emphasis on subsurface/geologic site characterization, evaluation of soil shear strength for static and seismic analysis, and seismic slope stability analysis procedures.

1.2 APPLICABLE REGULATIONS AND LAWS

The State of California currently requires analysis of the seismic stability of slopes for certain projects. Most counties and cities in southern California also require analysis of the static stability of slopes for most projects. The authority to require analysis of seismic slope stability is provided by the Seismic Hazards Mapping Act of 1990, which became California law in 1991 (Chapter 7.8, Sections 2690 et. seq., California Public Resources Code). The purpose of the Act is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure; or other hazards caused by earthquakes. The Seismic Hazards Mapping Act is a companion and complement to the Alquist-Priolo Earthquake Fault Zoning Act, which addresses only surface fault-rupture hazards. Chapters 18 and 33 (formerly 70) of the Uniform/California Building Code provide the authority for local Building Departments to require geotechnical reports for various projects.

Special Publication 117 (SP 117), by the California Department of Conservation, Division of Mines and Geology in 1997, presents guidelines for evaluation of seismic hazards other than surface fault-rupture and for recommending mitigation measures. The guidelines in SP 117 provide, among other things, definitions, caveats, and general considerations for earthquake hazard mitigation, including seismic slope stability.

SP 117 provides a summary overview of analysis and mitigation of earthquake induced landslide hazards. The document also provides guidelines for the review of site-investigation reports by regulatory agencies who have been designated to enforce the Seismic Hazards Mapping Act.

However, Building Officials from both the City and County of Los Angeles desired to have more definitive guidance to aid their agencies in the review of geotechnical investigations that must address seismic hazards and mitigations. Specifically, both agencies sought assistance in the development of recommendations for dealing with earthquake-induced liquefaction and landslide hazards. The City and County of Los Angeles were joined by their counterparts in other southern California counties that include Orange, San Bernardino, San Diego, Riverside, and Ventura counties.

Two "Implementation Committees" have been convened under the auspices of the Southern California Earthquake Center (SCEC) at the University of Southern California. The first addressed the issue of liquefaction, and liquefaction implementation guidelines were published in March 1999. This report is the product of the second committee on landslide hazards. The Landslide Hazards Committee has participating members from the practicing professional, academic, and regulatory communities.

The purpose of this document is two-fold. The first objective is to present information that will be useful and informative to Building Officials so that they can properly and consistently review and approve geologic and geotechnical reports that address slope stability hazard and mitigation. The second objective is to provide a broad-brush survey of some of the most common methods of analyses and mitigation techniques that will be useful to geotechnical engineers, engineering geologists, Building Officials, and other affected parties.

It is definitely not the intention of the Implementation Committee that this document becomes a set cookbook approach to evaluating slope stability hazard and mitigation. The changes and advances in geotechnical engineering and engineering geologic technology are occurring rapidly. An intent of this document to encourage the use of advanced yet proven technologies, so that sound hazard evaluations are performed.

This document presents information developed by the Implementation Committee that has been studied, debated, and agreed to by a consensus of the members. In general, the views presented in this document represent the unanimous opinion of the Committee members. On the topic of seismic slope stability analysis, however, it was not possible to reach consensus. There was a great deal of debate regarding the use of a seismic displacement method, because such a method represents a change from current practice. Several officials from regulatory agencies expressed concern that their agencies may resist change from the current practice because of the low frequency of seismically induced slope failure (i.e., the design event for an area occurs infrequently). In addition, they are uncertain what level of displacement their specific agencies will be willing to accept relative to habitable structures and other improvements. The results of the Committee's extensive deliberations are presented in Chapter 11. The analysis methods presented in Chapter 11 represent the consensus recommendations of all practicing and academic members of the Committee (regulatory officials chose not to vote). The Committee was unable to reach consensus on acceptable seismic slope displacements, and therefore regulatory agencies will need to establish their own values for this important parameter.

The Committee actively sought input from professional and academic sources across the U.S., and this report reflects the valuable input from those individuals.

1.3 LIMITATIONS

Ground deformations under static and seismic conditions can result from a variety of sources, including shear and volumetric straining. This report focuses on slope stability and seismic slope displacements, both associated with shear deformations in the ground. Ground deformations associated with volume change, such as hydrocompression or consolidation under long-term static conditions or seismic compression during earthquakes, are not covered by the actions of this committee. In addition, ground displacements associated with post-seismic pore pressure dissipation in saturated soil, or lateral spread displacements in liquefied ground, are not covered.

The intent of this report is to present practical guidelines for static and seismic slope stability evaluations that blend state-of-the-art developments in methodologies for such analyses with the site exploration, sampling, and testing techniques that are readily available to practicing engineers in the southern California area. Accordingly, the intent is not necessarily to present the most rigorous possible procedures for testing the shear strength of soil and conducting stability evaluations, but rather to suggest incremental rational modifications to existing practice that can improve the state-of-practice. It should be noted that the Committee by no means intends to discourage the use of more sophisticated procedures, provided such procedures can be demonstrated to provide reasonable solutions consistent with then-current knowledge of the phenomena involved.

2 ESTABLISHMENT OF "EARTHQUAKE-INDUCED LANDSLIDE HAZARD ZONES"

The Seismic Hazards Mapping Act of 1990 requires the State Geologist to delineate "seismic hazard zones," for various earthquake hazards, including earthquake-induced landslides. Criteria used to delineate Earthquake-Induced Landslide Zones were developed by the Seismic Hazards Mapping Act Advisory Committee for the California State Mining and Geology Board in 1993, and are contained in a revised document titled "Recommended Criteria for Delineating Seismic Hazard Zones in California" (CDMG, 1999). According to those criteria, Earthquake-Induced Landslide Hazard Zones are areas meeting one or more of the following:

1. Areas known to have experienced earthquake-induced slope failure during historical earthquakes.

2. Areas identified as having past landslide movement, including both landslide deposits and source areas.

3. Areas where CDMG's analyses of geologic and geotechnical data indicate that the geologic materials are susceptible to earthquake-induced slope failure.

Delineation of earthquake-induced landslide zones under criterion 3 is based on a Newmark (1965) methodology modified by the following assumptions:

1. The type of failure assumed is an infinite-slope; that is, a relatively shallow block slide that has a failure surface parallel to the ground surface.

2. Only unsaturated slope conditions are considered.

3. The response of the geologic materials to earthquake shaking, in terms of landslide failure potential, can be adequately characterized by the shear strength parameter, tan ϕ, for various geologic materials.

Adverse bedding conditions (out-of-slope bedding) and shear strength values representing the weaker materials (such as shale interbeds in a predominantly sandstone formation) within the mapped geologic unit are considered in the rock-strength grouping. If geotechnical shear test data are insufficient or lacking for a mapped geologic unit, the unit is grouped with lithologically and stratigraphically similar units for which shear strength data are available.

Based on calibration studies (McCrink, in press), hillslopes exposed to ground motions that exceed the yield acceleration for instability, and are associated with displacements greater than 5 cm are included in Earthquake-Induced Landslide Zones. The ground motion parameters used in the analysis include mode magnitude, mode distance, and peak acceleration for firm rock. Expected earthquake shaking is estimated by selecting representative strong-motion records, based on estimates of probabilistic ground motion parameters for levels of earthquake shaking having a 10 percent probability of being exceeded in 50 years (Petersen et al., 1996).

Seismic Hazard Zones for potential earthquake-induced landslide failure are presented on 7.5-minute quadrangle sheet maps at a scale of 1:24,000. Supplementary maps of rock strength, adverse bedding, geology, ground motions, and an evaluation report describing strength classification, Newmark displacements and regional geology and geomorphology are also provided for each quadrangle as the basis for delineation of the zones. The zone maps do not identify other earthquake-triggered slope hazards including ridge-top spreading and shattered ridges. Run-out areas of triggered landslides may extend outside the landslide zones of required investigation.

Seismic Hazard Zone maps are being released by the California Department of Conservation, Division of Mines and Geology. The maps present zones of required investigation for landslide and liquefaction hazards as determined by the criteria established by the Seismic Hazards Mapping Act Advisory Committee.

3 ROLES OF ENGINEERING GEOLOGISTS AND GEOTECHNICAL ENGINEERS

The investigation of the static and seismic stability of slopes is an interdisciplinary practice. The following paragraph has been extracted from Special Publication 117 regarding the roles of engineering geologists and geotechnical engineers.

California's Seismic Hazard Mapping Act and Regulations state that "The site investigation report must be prepared by a certified engineering geologist or registered civil engineer, who must have competence in the field of seismic hazard evaluation and mitigation, and be reviewed by a certified engineering geologist or registered civil engineer, also competent in the field of seismic hazard evaluation and mitigation. Although the Seismic Hazard Mapping Act does not distinguish between the types of licensed professionals who may prepare and review the report, the current Business and Professions Code (Geologist and Geophysics Act, Section 7832; and Professional Engineers Act, Section 6704) restricts the practice of these two professions. Because of the differing expertise and training of engineering geologists and civil engineers, the scope of the site investigation study for a project may require that professionals from both disciplines prepare and review the report, each practicing in the area of his or her expertise. For the purpose of the following discussion, an engineering geologist is defined as a Certified Engineering Geologist, while a geotechnical engineer is defined as either a Civil Engineer with expertise in soil engineering or a Geotechnical Engineer.

Involvement of both engineering geologists and geotechnical engineers will generally provide greater assurance that the hazards are properly identified, assessed and mitigated."

The Committee provides the following additional comments and guidance concerning appropriate professional practice with respect to the analysis of slope stability. Implicit within the following comments is the requirement that work be performed only by or under the supervision of licensed professionals who are competent in their respective area of practice. An engineering geologist should investigate the subsurface structure of hillside areas. The engineering geologist should provide appropriate input to the geotechnical engineer with respect to the potential impact of the subsurface geologic structure, stratigraphy, and hydrologic conditions on the stability of the slope. The assessment of the subsurface stratigraphy and hydrologic conditions of sites underlain solely by alluvial materials may be performed by the geotechnical engineer. The shear strength and other geotechnical earth material properties should be evaluated by the geotechnical engineer. The geotechnical engineer should perform the stability calculations. The ground motion parameters for use in seismic stability analysis may be provided by either the engineering geologist or geotechnical engineer, or a registered geophysicist competent in the field of seismic hazard evaluation.

4 SITE INVESTIGATION AND GEOLOGIC STUDIES

Literature review and field exploration are routinely performed for new projects as part of the normal design and development process. Geologic mapping and subsurface exploration are normal parts of field investigation. Samples of the earth materials are obtained during subsurface exploration for testing in the laboratory to determine the shear strength parameters and other pertinent properties.

Thorough geologic studies are a critical component in the evaluation of slope stability. Failures of "engineered" slopes can often be traced to inadequacies in geologic review and exploration (Slosson and Larson, 1995) such as failure to review aerial photographs, inadequate subsurface exploration, insufficient testing, and/or poor-quality analysis of available data. Adequate evaluation of slope stability for a given site requires thorough and comprehensive geologic and geotechnical studies. However, on rare occasions, slopes are constructed in areas where geologic conditions are known to be non-problematic from previous onsite subsurface exploration. An engineer may cite the existence of previous, site-specific geologic data as justification for not performing subsurface exploration. It is the responsibility of the engineer to demonstrate that the previous geologic studies are sufficient for the required stability analysis and to take responsibility for their proper use on the present project. Where the engineer cannot demonstrate the adequacy of prior work, the performance of geologic studies is required.

In general, geologic studies for slope stability can be broken into four basic phases:

1. Study and review of published and unpublished geologic information (both regional and site specific), and of available stereoscopic and oblique aerial photographs.

2. Field mapping and subsurface exploration.

3. Analysis of the geologic failure mechanisms that could occur at the site during the life span of the project.

4. Presentation and analysis of the data, including an evaluation of the potential impact of geologic conditions on the project.

Geologic reports should demonstrate that each of those phases has been adequately performed and that the information obtained has been considered and logically evaluated. Minimum criteria for the performance of each phase are described and discussed below.

4.1 BACKGROUND RESEARCH

The purpose of background research is to obtain geologic information to identify potential regional geologic hazards and to assist in planning the most effective surface mapping and subsurface exploration program. The availability of published references varies depending upon the study area. Topographic maps at 1:24,000 scale are available for all of California's 7.5' quadrangles. More detailed topographic maps are often available from Cities or Counties. Most urban locations in California have been the subject of regional geologic mapping projects. Other maps that may be available include landslide maps, fault maps, depth-to-subsurface-water maps, and seismic hazard maps. Seismic slope stability hazard maps prepared by the California Division of Mines and Geology (CDMG) are particularly relevant, and the location of a site within in a seismic slope stability hazard zone will generally trigger the type of detailed site-specific analyses that are the subject of this report. The above maps are typically published by the United States Geological Survey (USGS), CDMG, Dibblee Geological Foundation, and local jurisdictional agencies (e.g., Seismic Safety elements of cities and counties). Collectively, these maps provide information useful for planning a geologic field exploration. In addition, the maps provide insight into regional geologic conditions (and possible geologic constraints) that may not be apparent from focused site studies.

Review of unpublished references also should be a part of geologic studies for slope stability. Previous geologic and geotechnical reports for the property and/or neighboring properties can provide useful data on stratigraphy, location of the groundwater table, and shear strength parameters from the local geologic formations. Strength data should be carefully reviewed for conformance with the sampling and testing standards discussed in sections 6 and 7 before being used. Critical review of topographic maps prepared in conjunction with proposed developments can reveal landforms that suggest potential slope instability. These materials are usually kept by the local jurisdictional governing agency, and review of their files is recommended.

Once review of available geologic references has been performed, aerial photographs of the area should be reviewed. Often, the study of stereoscopic aerial photographs reveals important information on historical slope performance and anomalous geomorphic features. Because of differences in vegetative cover, land use, and sun angle, the existence of landslides or areas of potential instability is sometimes visible in some photographs, but not in others. Therefore, multiple sets of aerial photographs going as far back in time as possible should be reviewed to identify landslides or fault zones. Geologic reports for slope stability should demonstrate that such efforts have been adequately completed. Geologic reports should include discussions of the results of aerial photographic review, and relevant findings should be illustrated on topographic maps and grading plans for the proposed development.

4.2 FIELD MAPPING AND SUBSURFACE INVESTIGATION

The purpose of field mapping and subsurface exploration is to identify potentially significant geologic materials and structures at the site, and to provide samples for detailed laboratory characterization of materials from potentially critical zones. Surface mapping should be conducted of outcrops on the site and accessible outcrops in the vicinity of the site. Subsurface investigation is almost always required, and may be performed by a number of widely known techniques such as bucket-auger borings, conventional "small-diameter" borings, cone penetration testing (CPT), test pits, or geophysical techniques. The planning of a particular exploration program should consider the results of background research for the site (Section 4.1) and the needs of the proposed project.

Particular geologic features that may be sought based on background research are fault zones, slip surfaces for existing landslides, or adversely oriented geologic structures such as bedding planes. Identification of fault rupture hazards is not the subject of this report, but because faults can create zones of weakness, their presence should be considered. If a landslide is thought to be present that may impact the project, determination of the location of sliding surface(s) is vital. Locating slide planes generally requires continuous logging, which may be performed by coring, downhole logging of bucket-auger holes, or CPT soundings. If CPT soundings are used, a suspected slide plane should be confirmed with samples from nearby boreholes (which may be most conveniently performed after completion of the CPT).

Even if no adversely oriented geologic features such as faults, bedding fractures, or landslides are identified during background research, it is still possible that weak zones of significance to the project exist at the site. Subsurface exploration should be carried out to identify, determine the extent of, and sample such zones, if they exist. If no such zones are thought to exist, the investigation results must be sufficiently detailed to support that hypothesis. If the investigation is not of sufficient detail or quality to confirm the non-presence of such zones, then presumptive strengths (Section 7.3.1) should be assumed at the most disadvantageous location(s) and orientation(s) within the slope.

Borings and trenches, coupled with surface mapping are used to estimate the three-dimensional geometry of the critical geologic structure. Selection of the number, location, and depth of borings are critical decisions in slope stability studies that should be carefully considered before "going into the field." The number of borings required is a function of the areal extent of the development, available information from previous investigations, and the complexity of the geologic features being investigated. Sound geologic and engineering judgment is required to estimate the number of borings required for a specific site. Guidelines on minimum level of exploration necessary for various types of construction are presented in NAVFAC 7.01 (1986). In general, it is anticipated that the number of borings/trenches should not be less than three. Additional borings will be required in many cases when the geology is complex. Borings should be positioned such that extrapolation of geologic conditions is minimized within the areas of interest.

The depth of borings and test pits should be sufficient to locate the upper and lower limits of weak zones potentially controlling slope stability. It should be noted that movement of landslides can be accommodated across multiple slip surfaces. Accordingly, locating the shallowest potential slide plane at a site may not be sufficient. In general, the depth of exploration should be sufficiently deep that the static factor of safety of a slip surface passing beneath the maximum depth of exploration and through materials for which appropriate presumptive strength values are assumed is greater than 1.5.

As noted above, continuous logging of subsurface materials is generally required to locate zones of potential weakness. Downhole logging is commonly practiced in southern California, and is widely thought to be the most reliable procedure. Downhole observation of borings provides an opportunity for direct sampling of potentially critical shear zones or weak clay seams. Such sampling and subsequent laboratory testing can be used to estimate strengths along potential slip surfaces. Prevailing conditions such as the presence of subsurface water, bad air, or caving soil may make it unsafe or impractical to enter and log exploratory borings. In those circumstances, it is necessary to utilize alternative methods such as continuously cored borings, conventional borings with continuous sampling, or geophysical techniques. Although those methodologies may be useful, the data obtained from them have limitations as geologic conditions are inferred rather than directly observed. Therefore, when such methods are utilized, the limitations should be compensated for by more subsurface exploration, more testing, more conservative data interpretation, and/or more comprehensive engineering analysis.

Detailed and complete logs of all subsurface exploration should be provided in geologic reports. Written descriptions of field observations should be accompanied by graphic logs that depict the geologic units, subsurface water conditions at the time of drilling and any subsequent measurements, and information relevant to soil sampling (e.g., sampler used, driving system, blow count, etc.) (ASTM D1586 and D6066-98).

The stability of cut- or fill-slopes can be affected by the extent to which the exposed materials have weathered or will weather during the design life of the project. Slopes that have performed adequately for years can experience surficial- and/or gross-failures because of weathering-induced strength reduction. Consequently, the effect of weathering on the long-term performance of slopes should be considered and evaluated during site exploration.

Adequate evaluation of the effects of weathering requires that both the extent and depth of weathering, and its effects on the physical properties/strengths of the materials, be evaluated. The depth or extent to which a slope might weather depends upon the composition and texture of the earth materials and the climate to which it will be exposed. Permeable, highly fractured or faulted materials are likely to weather more rapidly and to a greater extent than intact, impermeable materials. However, relatively impermeable expansive soil can experience deep weathering, if they are subject to repeated cycles of wetting and drying. Wet climates tend to induce more weathering than do dry climates. The geologist should provide an estimate of the depth of weathering. Preferably, this estimate will be based on exploration of natural slopes or cut slopes which have been in existence for a period of time approximately equal to the design life of the proposed project.

Quantitative evaluation of the effects of weathering on the strengths of an earth material can be a difficult task as discussed in Chapter 7. Although weathering generally results in a reduction of strength due to mechanical de-aggregation and chemical decomposition, the amount of that reduction is difficult to quantify. Generally, the strength reduction can be estimated by testing samples of similar origin that have already been weathered in nature (e.g., a residual soil from a similar bedrock). Therefore exploration should be planned such that samples of weathered materials can be obtained for testing.

4.3 EVALUATION OF DATA

Once the data gathering portions of a geologic study have been completed, compilation and interpretation of such data are required. The results of these efforts should be illustrated on a composite geologic map and critical geologic cross-sections. Cross sections should be provided through the entire slope upon which the proposed development is to be situated. Reasonable interpolation of geologic structure between boreholes encountering similar geologic media is acceptable in the development of cross sections. To keep extrapolation beyond the geometric limits of investigation to a minimum, it may be necessary to obtain data from areas outside of the boundaries of a specific project.

The cross section(s) should show an interpretation for the entire slope based on the surface mapping, subsurface exploration, and regional geologic maps. The cross sections should show surface topography, locations of borings from which geologic structure is interpreted, existing landslide slip surfaces, and lines that represent interpretation of bedding planes, joints, or fractures. Sections that clearly show interpretation of geologic structure are necessary for subsequent engineering evaluation of stability because the ultimate determination of potential failure planes for analyses is dependent upon the accuracy of those sections. Because geologic structure is so critical to the evaluation of slope stability, potential modes of failure should be identified by the geologist, and evaluation of the most critical modes of failure should be a made by both the geologist and geotechnical engineer.

5 SUBSURFACE WATER

Subsurface water, if present in a slope or if it could develop during the life of a project, should be considered in slope stability analyses. The presence of subsurface water in a slope can reduce effective stresses when positive pore-water pressures develop, causing a reduction in shear resistance. Subsurface water can also increase de-stabilizing forces in the slope via the additional weight associated with a moist slide mass or via seepage forces. Therefore, engineers and geologists should investigate the presence of subsurface water and evaluate potentially adverse future subsurface water conditions.

Because the effects of subsurface water are critical to the ultimate stability of a slope, evaluation and interpretation of subsurface water conditions deserves careful consideration. Land development with its associated irrigation, and in some situations private sewage disposal along with seasonal rainfall variation, can result in significant changes in prevailing subsurface water conditions. Often, such changes are adverse and can significantly affect stability. Maximum subsurface water levels associated with extreme winter storm events coupled with irrigation sources should form the basis of static slope stability evaluations. Typical subsurface water conditions, accounting for normal seasonal rainfall patterns, should be employed for seismic slope stability evaluations. For either case (static or seismic), the post-development subsurface water level used in the analysis may be higher that that measured in the field at the time of drilling.

The future subsurface water level will depend on a number of geotechnical and hydrological factors, including soil permeability, geology, original position of the subsurface water level, intensity and duration of rainfall, amount of antecedent rainfall, rate of surface irrigation, rate of evapotranspiration, rate of waste water disposal, and subsurface flow from adjacent areas. Water levels for use in design can be estimated from piezometric data when sufficient, appropriate data are available. Analytical models together with conventional subsurface water-modeling techniques can provide reasonable estimates of future subsurface water levels.

As discussed by Duncan (1996), analyses of slope stability with a subsurface water level located above a portion of the sliding surface can be performed one of two ways:

1. By the use of total unit weights and specification of groundwater table location and boundary water pressures. This method is appropriate for effective stress analyses of slope stability and should be used with effective stress strength parameters. [If a total stress analysis is desired, it should be performed with no phreatic surface (i.e., zero pore pressure). Seepage forces should not be included. Total stress strength parameters should be used.]

2. By the use of buoyant unit weights and seepage forces below the water table. This method is appropriate for use only with effective stress analyses; it should not be used with total stress analyses.

Method 1 is most commonly selected. In a stability analysis utilizing Method 1, pore-water pressures are commonly depicted as an actual or assumed phreatic surface or through the use of piezometric surfaces or heads. The phreatic surface, which is defined as the free subsurface water level, is the most common method used to specify subsurface water in computer-aided slope stability analyses. The use of piezometric surfaces or heads, which are usually calculated during a seepage or subsurface water flow analysis, is generally more accurate, but not as common. Several programs will allow multiple perched water levels to be input within specific units through the specification of piezometric surfaces.

6 SAMPLING OF SOIL AND SOFT ROCK MATERIALS

6.1 GENERAL CONSIDERATIONS

It was noted in Section 1.0 that soil shear strength is a function of, among other factors, the stress history and density of the soil. Both of these factors influence the degree to which soil undergoes a contractive or dilatent response to applied shear, which strongly influences soil strength and stress-deformation response. What is significant about these factors from the standpoint of soil sampling is that they may be lost as a result of sample disturbance, causing the properties of laboratory specimens to deviate from those of in situ soil (Ladd and Foott, 1974). Therefore, the degree to which these factors are adequately represented in strength testing is a function of the sample disturbance associated with sampling procedures.

The significance of soil sampling in strength evaluations lies in the fact that soil is subjected to shear deformations and unloading during the sampling process. Therefore, sampling significantly changes the stress history of a soil sample. After sampling, many samples have the opportunity to drain or swell, which changes pore pressures and creates further changes in the stress history, over-consolidation ratio, and density of the sample relative to its prior in-situ state. This, in turn, causes the strength and stress-deformation response of the laboratory specimen to deviate from that of the in-situ soil. Some shearing during sampling is an unavoidable consequence of the unloading that occurs upon removal of the sample from the ground. However, different sampling procedures can impose a wide variety of additional shear strains on the soil sample, and those effects should be considered in the specification of a sampling method for a particular soil.

As an example, the shearing imparted during sampling of a contractive soil (i.e., a soil that will tend to decrease in volume when sheared under drained conditions) will either: (1) increase the density of the soil if it is unsaturated, or (2) increase the pore pressures in the sample if it is poorly drained and saturated. If the sample is subsequently subjected to a standard drained shear test, any excess pore pressures will be allowed to dissipate, and the tested specimen will be denser, therefore, stiffer and stronger than the in-situ soil. The converse is also true, namely a dilatant sample will decrease in density as a result of the sampling process; therefore, the tested specimen will be weaker than the in-situ soil.

6.2 SELECTION OF AN APPROPRIATE SAMPLING TECHNIQUE

It follows from the above reasoning that the sampling techniques that impart the least shear strain to the soil are most desirable. Commonly available sampling techniques include: (1) driven thick-walled samplers advanced by means of hammer blows, (2) pushed thin-walled tube samplers advanced by static force, and (3) hand-carved samples obtained from a bucket-auger hole or test pit.

Two types of thick-walled driven samplers are most often used in practice: (1) Standard Penetration Test (SPT) split spoon samplers, which have a 2.0-inch outside diameter and 5/16-inch wall thickness, and (2) so-called California samplers, which typically have a 3.0- to 3.3-inch outside diameter, 1/4- to 3/8-inch wall thickness, and internal space for brass sample tubes (which typically are stacked in 1.0-inch increments).

Pushed thin-walled tube samplers are typically 3 to 5 inches in diameter with an approximately 1/16 to 1/8-inch-thick walls. When configured with a 3.0-inch outside diameter and advanced with a simple static force, they are referred to as Shelby tubes (ASTM D1587). A sampler that provides less sample disturbance than Shelby tubes is a Hydraulic Piston Sampler (e.g., Osterberg type). It is often not possible to penetrate cohesionless soil or stiff cohesive soil with Shelby tubes, and in such cases a Pitcher tube configuration can be used. The sample tube used in a Pitcher tube sampler is identical to a Shelby tube, but the tube is advanced with the combination of static force and cutting teeth around the outside tube perimeter, which descend to the base of the tube when significant resistance to penetration is encountered.

Hand-carved samples are generally retrieved by removing an intact block of soil, which is transported to the laboratory. The sample is carefully trimmed in the laboratory to the size required for testing. Disturbed bulk samples can also be hand collected for remolding in the laboratory.

The selection of a sampling method for a particular soil should take into consideration the disturbance associated with field sampling as well as transportation and laboratory sample handling. Tube samplers require specimen extrusion and trimming, whereas the brass rings used in California samplers can be directly inserted into direct shear or consolidation testing equipment.

Specimens from SPT samplers are massively disturbed and should not be used for strength testing. Although the Committee is unaware of research documenting disturbance effects in California samplers, the tube thickness to area ratios associated with California samplers are such that a higher degree of disturbance would be expected than for thin walled tube samplers (e.g., Shelby tubes, piston tubes). Specimen extrusion and trimming in the laboratory are potentially significant additional sources of disturbance for specimens retrieved from both samplers.

The above factors make the selection of an appropriate sampler for a particular soil nontrivial. Nonetheless, some general guidelines can be provided:

1. The strength of clean granular soil (except gravel) is generally best estimated with correlations from normalized standard penetration resistance (SPT blow counts). CPT tip resistance values can be used to supplement, but should not replace, SPT blow counts for use in correlations. Blow counts from California samplers are not an acceptable substitute for SPT blow counts. If laboratory testing is desired in lieu of penetration resistance correlations, hand-carved samples (of cemented sand) or frozen samples are recommended. Samples of strongly dilatent soil (i.e., Pleistocene or older sand near the ground surface) obtained with a California sampler may be looser than the in-situ soil and, therefore, may provide a reasonably conservative estimate of soil shear strength.

2. Thick deposits of soft to firm clay (e.g., Holocene age clay such as San Francisco Bay Mud) should be sampled with pushed thin-walled tubes or a hydraulic piston sampler. Such soil is readily amenable to laboratory specimen extrusion. Hand-carved specimens are an acceptable substitute for tube samples.

3. Stiff to hard cohesive soil and clayey bedrock materials (claystone, shale) can be sampled with California samplers, Pitcher tube samplers, or could be cored. Soil strengths established from drained laboratory testing of such specimens are likely to be conservatively low with respect to in-situ conditions. Hand-carved specimens are a desirable substitute for tube and driven samplers.

4. Jointed or bedded bedrock often contains planes or zones of weakness, such as slickensided surfaces, gouge zones, discontinuities, relict joints, clay seams, etc., which control the strength and, therefore, the stability of the deposits. Sampling must be carefully performed so that the thin planes or zones of weakness are not missed. If brass ring samples are obtained in such materials, it is essential that the failure plane for a direct shear test be aligned with the planes of weakness. One desirable way to obtain such samples is to trim an area in the boring wall that is large enough for a standard, 1.0-inch tall brass ring. The ring can then be driven into the boring wall using a wood block and hammer. In other cases where critical zones are very thin, it may only be possible to retrieve bulk samples, which can

be cleansed of contaminating materials and remolded for subsequent testing in the laboratory (see Section 7.3.3(b)ii).

5. A conservative estimate of strengths along unweathered joint surfaces in rock masses can be obtained by pre-cutting in the laboratory an intact rock specimen and shearing the sample in a direct shear device along the smooth cut surface. The strength obtained from the pre-cut sample is generally a conservative estimate because actual joint surfaces have asperities not present in the lab specimen. Alternatively the rock may be repeatedly sheared without pre-cutting the sample. The objective in sampling for this type of testing is therefore an intact rock specimen, with the "joint" surface being created parallel to the direction of testing. Such samples can be obtained by coring, hand carving, or driving samples in non-brittle rocks.

6. Intact rock should be sampled by coring or hand carving to preserve sample integrity. California samples of intact rock will generally be fractured and significantly disturbed. Accordingly, shear strengths obtained from testing of specimens obtained with California samples will generally be lower than the actual strength of the in situ intact rock.

7. For new compacted fills, bulk samples of borrow materials can be obtained for re-molding and compacting in the laboratory.

8. Soil containing significant gravel generally can be sampled by hand carving of large specimens or correlations with penetration resistance can be used to estimate strengths. Correlations with penetration resistance are based on SPT blow counts or Becker penetrometer blow counts. Andrus and Youd (1987) describe a procedure to determine N-values in soil deposits containing significant gravel fragments. They suggest that the penetration per blow be determined and the cumulative penetration versus blow count be plotted. Changes in the slope of the plot indicate that gravel particles interfered with sampler penetration. Estimates of the effective penetration resistance of the soil matrix can be made for zones where the gravel particles did not influence the penetration.

6.3 SPACING OF SAMPLES

For most projects, samples from borings should be obtained at maximum 5-foot vertical intervals or at major changes in material types (whichever occurs more frequently). Samples in heterogeneous or layered materials should be obtained as often as needed to reflect the variability of the deposit and retrieve samples of the weakest materials that might influence slope stability. Larger sample-spacing intervals can be used for deep borings drilled primarily to obtain information on geologic structure

7 EVALUATION OF SHEAR STRENGTH

7.1 RECOMMENDED STRENGTH EVALUATION PROCEDURES

Soil behavior in shear is complex and depends strongly on drainage conditions, effective consolidation stresses prior to the onset of shear, the stress path followed by the specimen during shear (which, in turn, is a function of density and over-consolidation ratio, OCR, as discussed in 6.1), and strain rate.

In this section, we provide summary recommendations for evaluating shear strength for slope stability applications. Section 7.2 provided guidance on a number of critical decisions that must be made before assigning strength parameters for a slope stability analysis. These include:

1. Is the soil at the site likely to be critical under drained or undrained loading for static stability (Section 7.2.1)?

2. Should soil strength be characterized using effective or total stress strength parameters (7.2.1)?

3. If laboratory testing is performed to evaluate shear strengths, should peak, ultimate/fully softened, or residual strengths be used (7.2.2)?

4. Should laboratory-derived strength parameters be modified for rate effects (7.2.4)?

5. How can anisotropy and overburden effects on strength be incorporated into the evaluation of strength parameters (7.2.3 and 7.2.5)?

Each of these questions must be answered for an assessment of soil strengths for a slope stability analysis. Table 7.1 provides summary information for static strength selection for five commonly encountered conditions in California. Comments on the information in the table are provided below.

Table 7.1. Summary of Recommended Strength Evaluation Procedures

	Static Slope Stability					Seismic
Site Condition	**Drainage**	**Critical Stress Cond.**	**Strength Used**	**Rate Effects**	**Test Method**	
Fine-grained soft alluvium loaded by fill	Undrained	Total	Peak	Reduce peak strength by 30%	UTC (UU or CU) Vane Shear	Undrained, total stress, UTC (UU or CU), use judgment for pk. v. residual
Coarse-grained alluvium loaded or unloaded (unsaturated)	Drained	Effective	Peak	None	DDS, DTC	Effective Stress, drained, DDS, DTC
Coarse-grained alluvium, loaded or unloaded (saturated)	Drained	Effective	Peak	Check for liquefaction potential	DDS, DTC	Effective Stress, drained, DDS, DTC; use undrained residual strength if liquefiable
Saturated, fine-grained, overconsolidated, stiff alluvium or clayey bedrock with massive or supported bedding, Loaded	Undrained (check drained)	Total	Peak	Reduce peak strength by 30%	UTC	Undrained, total stress parameters, rate adjusted peak strengths
Unloaded	Drained	Effective	Depends on LL and CF	None	DDS, DTC (see Comment 3)	
Heavily overconsolidated saturated clay or clayey bedrock - pre-existing shear surfaces, loaded or unloaded	Drained	Effective	Residual	None	DDS, RS	Effective Stress, Drained DDS, RS

Commentary for Table 7.1

Comment (1): Soft clay is generally contractive when sheared, so undrained strengths are used, which are generally most conveniently represented with total stress strength parameters (use of effective stress strength parameters would require modeling of pore pressure response in situ). Analyses can be performed with peak strengths adjusted for rate effects, but if significant shear deformations are likely in the slope (even if the factor of safety exceeds 1.5), strengths between peak and residual should be selected.

Comment (2-3): Sandy soil with low fines-content (<15% plastic fines or non-plastic fines) is relatively free-draining and will typically be drained under static loading conditions. At low confining pressures, compacted sand may be dilatent and exhibit strain softening. Peak strengths can be used if significant shear deformations are not anticipated. If significant shear deformations are likely, residual strengths should be used in potentially dilatent sand.

Comments (4): Loading of highly overconsolidated, saturated alluvium or clayey bedrock could be critical under short-term undrained, or long-term drained loading (both should be checked). Undrained case need not be checked if material will be unsaturated. The effects of anisotropy and rate effects on undrained strength, and overburden pressure on drained or undrained strength, may be significant and should be considered.

Comment (5): Laboratory-derived strengths should be checked against published correlations (Fig. 7.5) and if a significant deviation is found, some justification should be provided. Residual strengths along a sliding surface are highly anisotropic (they can only be applied along the slip plane), and are somewhat sensitive to overburden pressure. Accordingly, testing should be performed at the overburden pressures expected in the field.

End of Commentary

For the rapid stress application that occurs during earthquake shaking, shearing occurs under undrained conditions. For that condition, the following types of strength parameters are recommended:

- Clay: Total-stress strength parameters from undrained test (CU or UU)
- Clay at residual: Effective-stress strength parameters, drained or undrained test
- Sand, unsaturated: Effective-stress drained strength parameters
- Sand, saturated: See below

For saturated sands, the pore pressure generated during shaking should be estimated with a liquefaction analysis. The undrained residual strength should be used if the soil liquefies, which can be estimated using available correlations with penetration resistance (i.e., Fig. 7.7 of Martin and Lew, 1999). A drained strength should be used if the soil does not liquefy, but the pore pressure generated during shaking should be estimated, so that the effective stress in the soil can be appropriately reduced.

The criteria in the "Seismic" column of Table 7.1 can be applied to the selection of strengths for seismic stability analyses. The principal comments associated with those criteria are as follows:

With respect to strain-softening effects, initial analyses can be performed with peak strengths. However, if slope displacement analyses indicate significant shear deformations in the slope, strengths should be reduced to values between peak and residual (depending on the soil characteristics and the amount of the computed displacement).

As discussed in Section 7.2.4, rate effects tend to increase the undrained strength of fine-grained materials, but may be partially offset by cyclic strength degradation effects.

7.2 GENERAL CONSIDERATIONS

7.2.1 Drainage Conditions and Total vs. Effective Stress Analysis

Soil behavior during drained loading is fundamentally different than during undrained loading. Drained loading implies that loads are applied at a sufficiently slow rate that no pore pressures are generated in the soil during shear, and volume change is allowed. Brinch-Hansen (1962) referred to this as "consolidated-drained" or CD loading, and that nomenclature will be used here. Undrained loading refers to a shear condition in which no volume change occurs, accordingly increased pore pressures will be generated in saturated, contractive soil, and decreased pressures in saturated, dilatent soil. Undrained shear can occur immediately after construction, or upon loading that follows consolidation of the soil. These cases are referred to as "unconsolidated-undrained" (UU) and "consolidated-undrained" (CU) loading by Brinch-Hansen (1962), respectively. Additional information about the use of CD, CU, and UU tests is available in Holtz and Kovacs (1981).

Once an appropriate drainage condition has been determined, the second major issue is whether effective or total stress strength parameters are to be used during the analysis. The strength of soil sheared under drained conditions (CD) is described with effective stress strength parameters. Using the Mohr-Coulomb failure criterion as illustrated in Fig. 7.1, the shear stress on the failure plane at failure (τ_{ff}) is taken as

$$\tau_{ff} = c' + \sigma_{f,f}' \tan\phi' \text{ (drained, CD)} \qquad (7.1a)$$

where c' and ϕ' are the effective stress cohesion intercept and friction angle, respectively. Effective stress $\sigma_{f,f}'$ = the effective normal stress on the failure plane at failure. Drained strength parameters are commonly evaluated using direct shear or triaxial apparatus. A schematic illustration of the stress states at failure from these tests is provided in Figure 7.1.

Obviously, the evaluation of parameters c' and ϕ' across a normal stress range of interest requires conducting multiple tests at different consolidation stresses, σ_c' (in the triaxial test) or different effective normal stresses, $\sigma_{f,f}'$ (in the direct shear test).

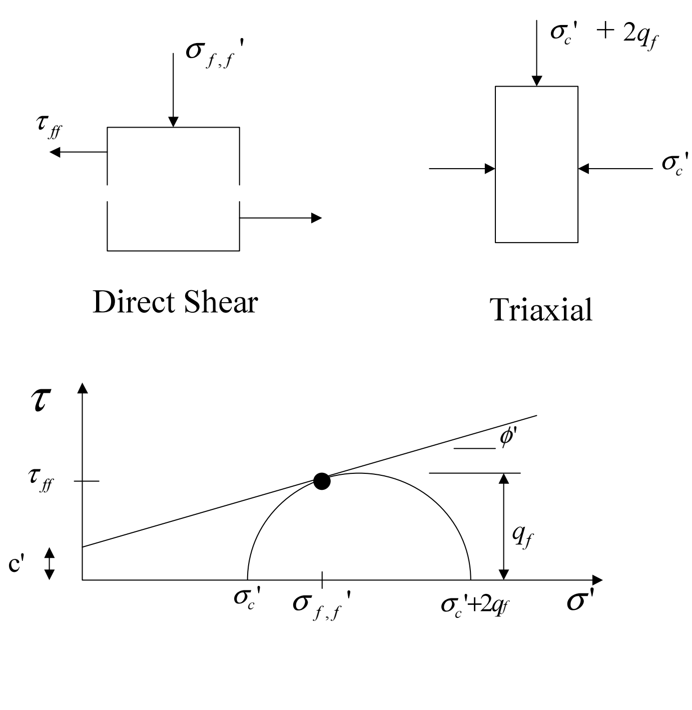


Figure 7.1. Stress States at Failure in Direct Shear and Triaxial CD Tests

The undrained shear strength of soil also can be described using effective stress strength parameters, but this is seldom done in routine practice because the use of such parameters in design would require an evaluation of pore-pressure response in the field during construction, which is a non-trivial analysis. Accordingly, shear strengths from UU or CU tests are typically defined using alternative strength parameters. End-of-construction (UU) strengths are described using conventional total stress strength parameters, i.e.,

$$\tau_{ff} = c + \sigma_{f,f} \tan\phi \text{ (end-of-construction, UU)} \tag{7.1b}$$

where σ_{ff} = total normal stress on the failure plane at failure. This linear approximation is only appropriate over a fairly short range of normal stresses. For saturated soil, ϕ=0 in Eq. 7.1b, and the strength is often denoted as $\tau_{ff} = s_u$ or $\tau_{ff} = c$. As illustrated in Fig. 7.2, these strength parameters are generally obtained with triaxial testing, as sample drainage cannot readily be controlled in direct shear tests. As indicated in the figure, triaxial tests are performed at a cell pressure σ_{cell}, and the shear strength τ_{ff} is obtained as half the deviatoric stress ($2q_f$).

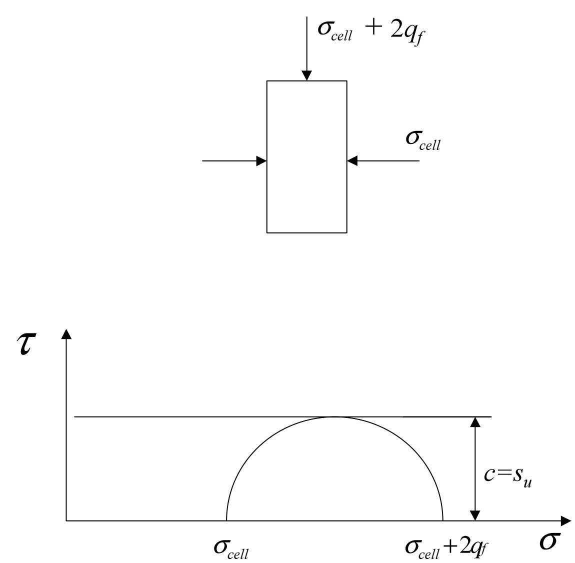


Figure 7.2. Stress State at Failure in Triaxial UU Test

As described by Casagrande and Wilson (1960) and Ladd (1991), post-consolidation, undrained (CU) strengths are evaluated by first consolidating the soil to a specified effective consolidation stress, σ_c', and then shearing the soil rapidly to failure. The shear stress on the failure plane at failure (τ_{ff}) is best evaluated by plotting the Mohr Circle in effective stress space, as shown previously in Figure 7.1. The shear strength of the soil, when evaluated in this manner, is typically found to be proportional to the consolidation stress σ_c',

$$\tau_{ff} = \sigma_c' \tan(\Psi_u) \text{ (consolidated-undrained, CU)} \tag{7.1c}$$

where $\tan(\Psi_u)$ is a constant that for a given soil mineralogy and structure depends only on OCR. As with UU tests, CU tests must generally be performed using a triaxial apparatus. Thus, when coupled with an OCR profile established from consolidation testing, values of $\tan(\Psi_u)$ can be used to evaluate profiles of equivalent total stress strength parameters s_u through a clay layer. This is accomplished by combining the effective consolidation stresses in the field that are present prior to the onset of shear with $\tan(\Psi_u)$ using Eq. 7.1c, where σ_c' is taken as the major principal effective stress in situ prior to the onset of shear.

Guidelines on the appropriate use of drained vs. undrained strength parameters are provided below. In the text, "loading" refers to a condition in which total normal stresses along potential sliding surfaces are increased as a result of the construction, for example the placement of fill, structural loads, etc. Conversely, "unloading" refers to a condition in which total normal stresses are decreased, such as excavations or rapid drawdown. In saturated soil, the total stress increase associated with loading tends to increase the pore pressures in the ground, whereas unloading reduces pore pressures. Pore pressures can also increase or decrease as a result of shearing, depending on whether the soil is contractive or dilatant. The guidelines are as follows:

1. Static loading of clean sand will generally be drained (i.e., CD). Soil strength should be represented with effective stress strength parameters.

2. Static loading of saturated clay with low OCR (OCR < 4) will be most critical under short-term undrained loading conditions (Mayne and Stewart, 1988, Ladd 1971). Examples of these materials include marine clay such as San Francisco Bay Mud, alluvial clay found in many valley areas in California, and clayey fill soils at depths > 10-20 ft (Duncan et al., 1991). These strengths can be represented with total stress strength parameters (UU) or with CU strength parameters [i.e., $\tan(\Psi_u)$, which is a function of OCR].

3. Static loading of heavily over-consolidated saturated clay (OCR > 4 to 8), including clayey bedrock materials, may be critical under short-term undrained or long-term drained conditions (CD). Heavily over-consolidated clay that is unsaturated under short-term conditions, but can be anticipated to become saturated, will generally be critical under long-term drained conditions.

4. Sand and stiff clay subject to shear as a result of unloading (e.g., cut slopes and other excavations) will be most critical under drained conditions (CD).

5. Unloading of soft clay may be critical under short-term undrained or long-term drained conditions. Strengths representative of both conditions should be evaluated for stability analyses.

For saturated or nearly saturated soils, rapid stress application during earthquake shaking occurs as undrained loading. Accordingly, either total stress or CU strength parameters should be used. If, prior to the probable earthquake, effective stresses in the soil can be expected to change with time due to consolidation, it may be reasonable to use CU strengths based on effective consolidation stresses that will be present in the slope after the completion of some acceptable amount of consolidation. Assuming the construction being analyzed involves loading of the ground, the range of effective possible consolidation stresses that could be chosen is, as a minimum, the effective consolidation stress prior to construction, and as a maximum, the effective consolidation stress after all excess pore pressures from loading have dissipated. The choice of which consolidation stress within this range should be used is project-specific, and should be selected after discussion between the consultant and regulatory official. Conversely, clayey soil subject to unloading will swell over time, and the reduced effective stresses present after the completion of swell should be used for seismic design.

Negative pore pressures are present in unsaturated soils. Limited experimental and centrifuge studies have shown that at saturation levels of 88% and 44%, these negative pore pressures may rise (i.e., become less negative) during rapid cyclic loading (Sachin and Muraleetharan, 1998; Muraleetharan and Wei, 2000). The available information is far from exhaustive, but those studies preliminarily suggest that at the pre-shaking saturation levels considered, the pore pressures can rise to nearly zero, but are unlikely to become positive. That behavior is less likely to occur in materials with higher degrees-of-saturation (for example, > 90%), because the relative scarcity of air bubbles could lead to the development of positive pore pressures. Accordingly, for materials that can be expected to have moderate saturation levels (< 90%), an assumption of zero pore pressure in the soil is likely to be conservative, meaning that stability analyses can be performed using effective stress strength parameters derived from drained shear tests. Those strength parameters should be used with effective stresses calculated for a zero pore pressure condition (i.e., effective stress = total stress).

7.2.2 Post-Peak Reductions in Shear Strength

All limit equilibrium methods for slope stability assume a rigid-perfectly plastic soil stress-deformation response, as depicted in Fig. 7.3. Because this model assumes strength to be independent of deformation, it can be difficult to apply to soil subject to post-peak reductions in shear capacity (i.e., soil with strength that is dependent on the level of deformation). Many soils experience such reductions, raising the question of which point along the stress-strain curve should be used to define the shear strength in a limit equilibrium model.

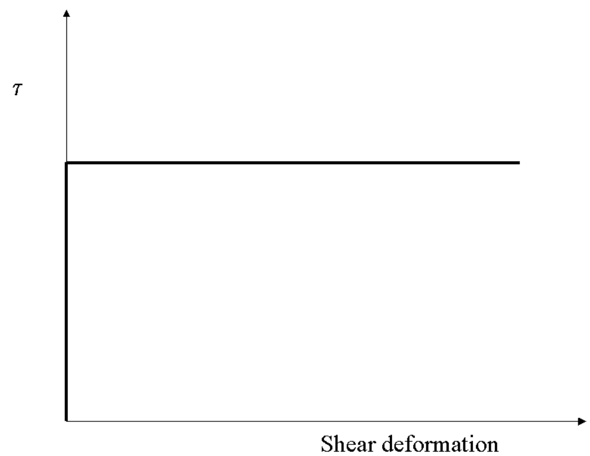


Figure 7.3. Depiction of Rigid Perfectly Plastic Soil Stress-Deformation Response

A typical stress-deformation curve for a clayey material under drained conditions is shown in Fig. 7.4. Skempton (1985) defined various points along the degrading stress-strain curve as follows. The maximum shear strength achieved for the sample after the initial nearly elastic behavior (Point A) is referred to as the "peak strength." The shear strength then drops to a post-peak value with additional deformation, marked by an inflection in the stress deformation curve (Point B) that is referred to as the "ultimate strength." The ultimate strength is achieved by an increase in moisture content (i.e., dilation) and to a lesser extent by particle re-orientation in clayey soil. Then, with a very large amount of deformation, the shear strength reduces to a nearly constant value (Point C) that is called the "residual strength." The "residual strength" is reached through re-orientation of clay particles in soil with a significant clay fraction. It should be noted at this point that the stress-deformation curve shown in Fig. 7.4 is a "backbone curve" enveloping multiple cycles of a direct shear test or results from a ring shear test. Details of how this backbone curve can be obtained from direct shear testing are presented in Section 7.3.3b.

Another strength term that will be referred to in this report is the "fully softened strength." As defined by Stark and Eid (1997), the fully softened strength is the peak strength obtained from a single cycle shear test performed on a reconstituted soil sample that is normally consolidated to the desired effective stress from a paste. Like the ultimate strength, the fully softened strength applies to a condition in which dilation is not contributing to soil strength, and particle reorientation effects are not yet fully realized. Accordingly, the two strength parameters are fundamentally identical. The distinction in terms is made here based on the means by which the strength is measured (i.e., intact specimen for ultimate; reconstituted specimen for fully softened).

The above strength terms are used in the context of drained shear. Undrained specimens can also experience strain softening, often due to pore pressure increase and/or particle re-orientation. For undrained shear, we will only refer to two strength values - peak and residual.

Skempton (1985) reports that fully softened/ultimate and residual drained shear strengths are approximately equivalent for materials with clay contents less than 25% (with clay defined as material finer that 0.002 mm). Drained residual strengths are less than fully softened strengths for materials with higher clay contents.

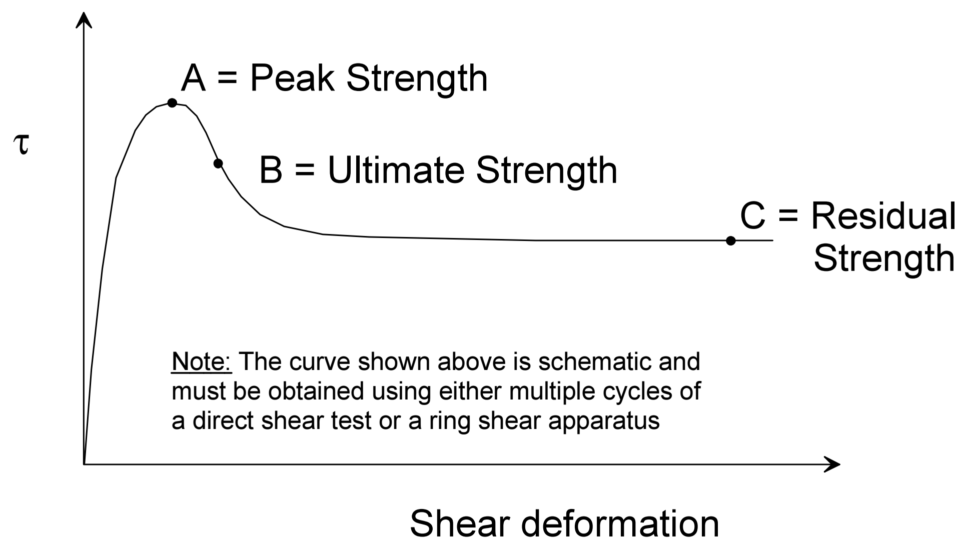


Figure 7.4. Diagrammatic Stress-Displacement Curve

Many materials can experience a post-peak reduction in strength, including most clayey soil (under drained or undrained conditions), dense sand under drained conditions, loose sand under undrained conditions, and cemented soil.

The following guidelines apply to the selection of appropriate strength parameters in materials subject to strain softening during long-term, drained loading conditions.

1. Residual strengths should be used in materials that have experienced significant previous shear deformations. Examples include materials located along pre-existing landslide slip surfaces and along continuous bedding planes likely to have been subject to significant past movement (e.g., folded bedrock that may have experienced flexural slip along bedding planes). Residual strengths should be used in those materials, even if the relative movement across the discontinuity occurred thousands of years ago (Skempton and Petley, 1967).

Residual strengths need not be used on joints or other discontinuity surfaces in fractured rock materials that have not experienced significant relative movements (Skempton and Petley, 1967). The deformations required to bring geologic materials to residual strength are generally thought to be on the order of several inches. However, Skempton and Petley (1967) have reported that displacements as low as approximately 5 mm can bring strengths to residual. Buried clayey residual soil (i.e., old topsoil) may also have reached residual strengths due to creep.

2. Peak drained strengths can be used for soil that is granular, non-plastic, and non-cemented. Peak drained strengths also can be used for crystalline bedrock materials that are unlikely to experience significant weathering over the project life.

3. Peak strengths can be used for fine-grained, low-plasticity materials (LL < 40) that have not experienced significant previous shear deformations, and are unlikely to be subject to significant weathering over the life of the project.

4. The strength of fine-grained, low-plasticity materials (LL < 40) that are likely to be subject to significant weathering should be measured using a mechanically de-aggregated sample to simulate the physical weathering process of the in situ soil. The peak strength from that test should be used.

5. Stiff clay and clayey bedrock materials (e.g., claystone, shale) of high plasticity (LL > 60) fail at shear stresses that are typically intermediate between the fully softened and residual strength (provided they had not been subject to significant previous shear deformations).

6. For stiff clay and clayey bedrock materials with LL = 40-60, strengths should be interpolated between the unadjusted peak value (corresponding to LL = 40) and the reduced value for strain softening effects (corresponding to LL = 60).

7. The selection of strengths for cemented, massive granular materials that break down when sheared beyond some threshold shear strain must be selected in consideration of the likely deformations that will occur in the field. The peak strength in such materials can only be reliably measured from testing of high quality "undisturbed" samples (defined in Item 6 in Section 6.2), whereas the residual strength can be defined from undisturbed samples at large deformations or more conventional, disturbed samples. Static and seismic stability analyses for those types of materials should be performed using both peak and residual strength parameters, and are discussed further in Section 9.1. In materials that are cemented and jointed (i.e., non-massive), judgment must be exercised on the use of strengths for unfractured cemented material vs. strengths along the joint surfaces. This decision should be based on a sound geologic assessment of geologic structure (i.e., see Section 4.2), likely

slope failure mechanisms at the site, and strain compatibility of shear strengths for materials along the failure surface.

Recommendations 3, 5, and 6 above are based on comparisons of mobilized shear strength (established from back analyses of first time slides) to fully softened and residual shear strengths by Stark and Eid (1997), and updated by Stark and McCone (2001). The Committee recognizes that ground conditions at the sites considered by Stark and Eid (1997) may not be directly comparable to materials that weather from older bedrock (pre-Quaternary). It is, however, the consensus of the Committee that these recommendations represent the best approach currently available. With respect to Recommendation 4 (weathered soil), the samples tested for Atterberg limits and shear strength should be taken from naturally weathered deposits of a similar earth material at or near the site. To distinguish between the levels of plasticity referred to above, visual classifications can be used in lieu of formal Atterberg Limits testing.

For undrained loading of clayey soil, Ladd (1991) found back-calculated values of $\tan(\Psi_u)$ from field case histories to be similar to laboratory CU test results adjusted for strain compatibility effects. The laboratory CU parameters for which these comparison were made represent peak strengths, hence, it is inferred that strain-compatibility adjusted peak strengths can be used for field applications. Strain compatibility adjustments to peak shear strength are discussed in Section 4.9 of Ladd (1991).

7.2.3 Soil Anisotropy

Stress and fabric induced anisotropy, as well as pre-existing shear zones, can lead to shear strengths that are dependent on the orientation of the failure plane. Slopes with pre-existing shear zones should be analyzed using along-bedding and across-bedding strengths applied to relevant portions of the failure surface (guideline #4 for sampling along bedding is included in Section 6.2).

For relatively homogeneous alluvial soil subjected to undrained loading, laboratory testing that shears samples across horizontal planes (such as triaxial tests on specimens retrieved from vertically advanced samplers) generally provide unconservatively high estimates of shear strength along the actual failure surface in the field (Duncan and Seed, 1966a and 1966b). Such effects are less significant for homogenous soil subjected to drained loading (Mitchell, 1993).

7.2.4 Rate Effects

Laboratory shear tests are generally performed over the course of minutes to days. Field loading under static loading is much slower, whereas seismic loading is more rapid.

Strength loss under static loading as a result of creep can be important in soft clay subjected to undrained loading and heavily over-consolidated clay (including claystone bedrock) in drained shear (Mitchell, 1993). Non-plastic materials and crystalline bedrock are not subject to creep strength loss. In clayey materials subject to undrained shear, creep can begin to occur when the shear stress exceeds about 50 to 70 percent of the peak shear strength, possibly leading to eventual failure. Therefore, it is recommended that, for sensitive clayey materials, the peak undrained strengths from laboratory testing conducted at "normal" strain rates be reduced by about 30 percent for use in static undrained analysis. However, the peak strength need not be reduced to a value less than the residual undrained strength. Testing by Skempton (1985) has shown that static residual strengths are relatively unaffected by strain rate; therefore, no reductions are needed for laboratory residual strengths.

Strength loss in clayey soil over long periods of time (i.e., drained conditions) has also been widely reported (Skempton, 1985; Mitchell, 1993). However, it is difficult to distinguish strength loss associated with negative pore pressure dissipation from strength loss from creep when clay is sheared under drained conditions. Since this issue does not appear to have been adequately resolved in the geotechnical engineering literature, the Committee does not recommend any adjustment to true drained strength parameters derived using the procedures described in this document.

For rapid (seismic) loading, testing should be performed under undrained loading conditions. Both the peak and residual shear strengths of fine-grained soil can be changed in undrained tests conducted at "rapid" strain rates relative to undrained tests conducted at "normal" strain rates. Peak shear strengths typically increase with increasing strain rate (e.g., Lefebvre and LeBoeuf, 1987; Ishihara et al., 1983; Dobry and Vucetic, 1987; Lefebvre and Pfender, 1996; Sheahan et al., 1996). Results of these investigations indicate that increases in dynamic strength relative to static undrained strengths can be on the order of 10-40%. While these effects can be partially offset by cyclic strength degradation effects, for most practical applications the undrained shear strength available to fine-grained sediments under seismic conditions are expected to be larger than those measured in typical laboratory tests. The effect of strain rate on drained residual strengths was investigated by Skempton (1985) and Lemos et al. (1985). Their results suggest that the residual strengths of clay-rich materials (> 50% clay content, e.g., claystone, shale) are generally higher for rapid strain rates (> 100 mm/minute) than for ordinary strain rates. However, their testing also suggests that the residual strength for materials with intermediate clay contents (approximately 25%) can decrease with increasing strain rate. It is not clear from these papers whether the observed variations in strength from tests conducted at different strain rates are in fact resulting from pore pressure generation or true strain rate effects. Further research is needed on this topic. It is the judgment of the Committee that, based on the current state of knowledge, the residual strength friction angle from a drained test conducted at "normal" strain rates can be used as a first-order approximation of the residual strength friction angle under undrained and rapid loading conditions.

7.2.5 Effect of Confining Stress on Soil Failure Envelope

The effect of confining stress on the stress-strain response of granular materials has been summarized by Lambe and Whitman (1969) as follows:

1. As confining pressure increases, the peak normalized shear strength (i.e., secant friction angle based on peak strength) decreases.

2. The fully softened/ultimate strength is more-or-less independent of changes in confining pressure.

The strong effect of confining pressure on normalized peak shear strengths has been attributed to a decreased tendency for dilation at large confining pressures, and a reduced level of grain interlocking (and increased grain crushing) as confining pressures increase (Lambe and Whitman, 1969; Terzaghi et al., 1996). This reduction of friction angle with increasing confining pressure causes downward curvature of the failure envelope.

For clayey soil, Skempton (1985) and Stark and Eid (1994) have found downward curvature of failure envelopes representing the residual strengths, and Stark and Eid (1997) have found downward curvature of failure envelopes for fully softened strength. Therefore, curvature of failure envelopes is an issue faced in both cohesive and cohesionless materials. At low confining pressures, curvature can be particularly pronounced, as failure envelopes for residual strength pass through or nearly through the origin

Given the above, it is important to perform shear strength testing across the range of normal stresses expected in the field. A curved representation of the failure envelope can be used in many modern computer programs, and is the preferred method for accounting for these effects. If this is not possible, a linear representation of the actual curved failure envelope can be used across the range of normal pressures expected in the field. It should be noted, however, that, in situations where both shallow and deep-seated stability must both be analyzed, more than one linear envelope would need to be established.

At sites with particularly deep-seated slip surfaces, it may not be possible to perform testing at the normal pressures occurring in the field. In such cases, testing should be performed across a range of lower normal stresses to establish the variation of friction angle with increased stress. This variation can be described in terms of power, cycloid, and hyperbolic equations (Duncan et al., 1989; Atkinson and Farrar, 1985; Maksimovic, 1989; Vyalov, 1986). These expressions can then be used to extrapolate the failure envelope beyond the tested range to the normal stresses expected in the field.

7.3 PROCEDURES FOR ESTIMATING SHEAR STRENGTH PARAMETERS

As described in Section 7.2, a rational analysis of soil shear strength begins with an assessment of whether shearing will occur under drained or undrained conditions. This assessment, coupled with knowledge of the soil/rock type, allows the engineer to select whether total or effective stress strength parameters are most appropriate for a particular soil. The effects of strain-softening, anisotropy, strain rate, and confining pressure also need to be taken into consideration when selecting shear strength parameters.

Once the conditions for which strength parameters will be used have been established, an appropriate method for evaluating them can be implemented. The following general procedures can be used to evaluate shear strength:

- Presumptive Values - Established locally by building departments.
- Published Correlations - Shear strength is related to another indicator test such as penetration resistance (SPT or CPT) or Atterberg Limits/Clay Fraction.
- In-Situ Measurements - Vane Shear (useful only to determine undrained strength).
- Laboratory testing - Determined by various tests including Direct Shear, Triaxial Compression, Triaxial Extension, Direct Simple Shear, Torsional Shear, and Ring Shear.
- Back Analysis - Determined mathematically by assuming that the slope has a factor of safety of 1.0. This approach is mostly used where failure has occurred (therefore, the factor of safety is known to have dropped below 1.0). The slope geometry, failure surface geometry, and groundwater conditions at the time of failure are utilized. Care should be exercised in applying back-calculated strength parameters to the analysis of slopes other than the failed slope. The consultant must demonstrate that similar earth materials are present at each location where the back-calculated strength is used. The method can also be used to obtain conservative strength values for a non-failed slope that is assumed to have a factor of safety of essentially 1.0.

Each of the above methods of strength evaluation are optimized for different loading conditions, and are consequently limited in their range of applications. In the sections that follow, each strength evaluation technique is described and its limitations outlined.

7.3.1 Presumptive Values

Conservative presumptive shear strength parameters can be used in slope stability analyses for sites where no field exploration or laboratory testing have been performed. Because these presumptive strength parameters are used in lieu of site-specific exploration or testing, they must be chosen conservatively, so that the probability that lower strength parameters exist at a site is very low. In general, presumptive values should be selected and approved by local regulatory reviewing agencies in a manner that incorporates data from local case histories, experimental data, and back analyses. These values apply only for the drainage conditions, loading rates, etc. that were present in the tests/case studies from which the values were derived. Provided they are used for a comparable set of conditions, presumptive strength parameters should yield a safe design, but not necessarily an economical one. For most projects, it should be economically beneficial to perform field exploration and laboratory testing to develop project-specific shear strength parameters rather than use low, presumptive strength values. It also should be noted that presumptive strength parameters are intended to be realistic lower bound strength values and are not intended to be lower than any values ever obtained.

7.3.2 Published Correlations

As described previously in Section 6.2, in most cases the drained strength of sand and non-plastic silt is best estimated by correlations with SPT blow count and CPT tip resistance. The recommended SPT correlation for sand is shown in Fig. 7.5a. Note that the blow count $[(N_1)_{60}]$ is corrected for procedure to 60% efficiency, and corrected to 1.0 atm overburden pressure. CPT tip resistance is also normalized to 1.0 atm overburden pressure in the correlation shown in Fig. 7.5b. SPT and CPT procedure and overburden correction factors are discussed in detail in Martin and Lew (1999).

Evaluation of the drained or undrained shear strength of clay should be accomplished with testing. However, it is good practice to check laboratory-derived strength parameters for clay using available correlations. A particularly onerous problem with clay strength evaluations can be the evaluation of residual shear strengths for thin failure surfaces. This problem arises principally from difficulty in sampling and properly orienting test specimens in direct shear devices. Accordingly, it is strongly recommended that sufficient clay be obtained by scraping the surface to allow determination of the liquid limit and clay fraction, so that the residual shear strengths for clay slip-surfaces can be checked using published correlations such as those by Stark and McCone, 2001 (updated from Stark and Eid, 1994 and 1997). Correlations between soil liquid limit and clay fraction (established by a ball-milling technique) and friction angle are shown in Figures 7.5c (residual friction angle) and 7.5d (fully softened friction angle). Care should be exercised when using these correlations because liquid limits and clay contents derived using ball-milled samples differ from those obtained using standard ASTM techniques. Figures 7.5e and 7.5f can be used to relate those ASTM and ball-milled index properties for use with the friction angle correlations in Figure 7.5c and 7.5d. Additional information on the interpretation of direct shear test results for residual strength is provided in the following section.

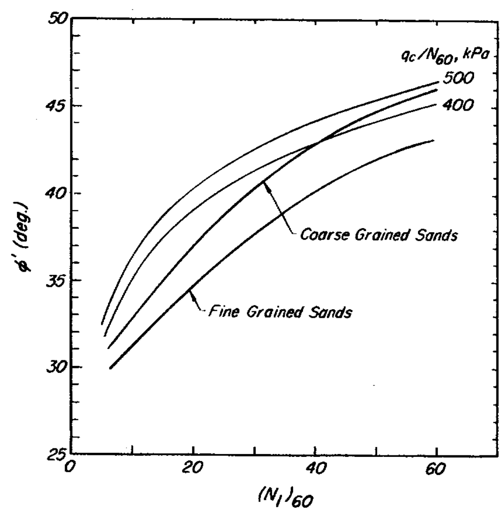


Figure 7.5a. Empirical Correlation Between Friction Angle of Sand and Normalized Standard Penetration Blow Count (Terzaghi et al., 1996)

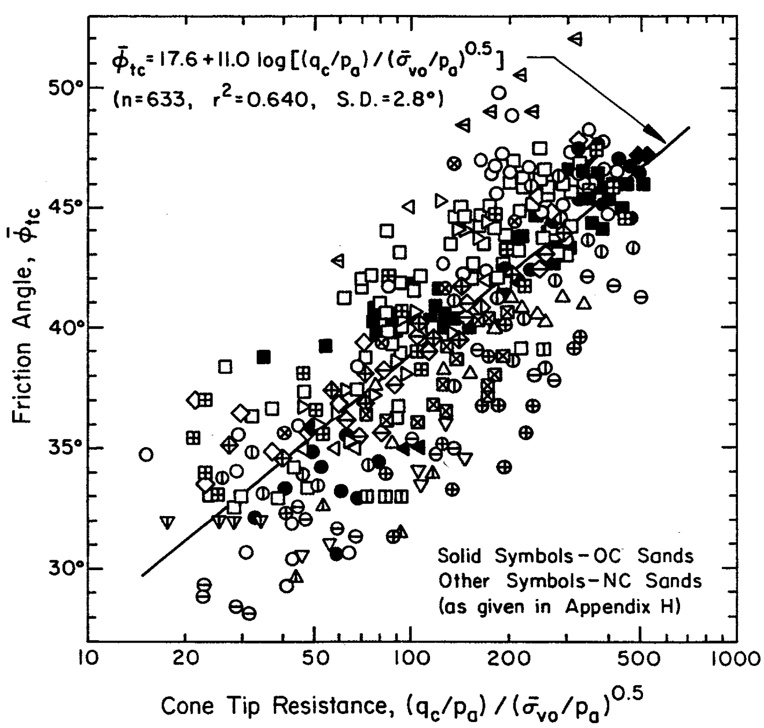


Figure 7.5b. Empirical Correlation Between Friction Angle of Sand and Normalized CPT Tip Resistance (Kulhawy and Mayne, 1990)

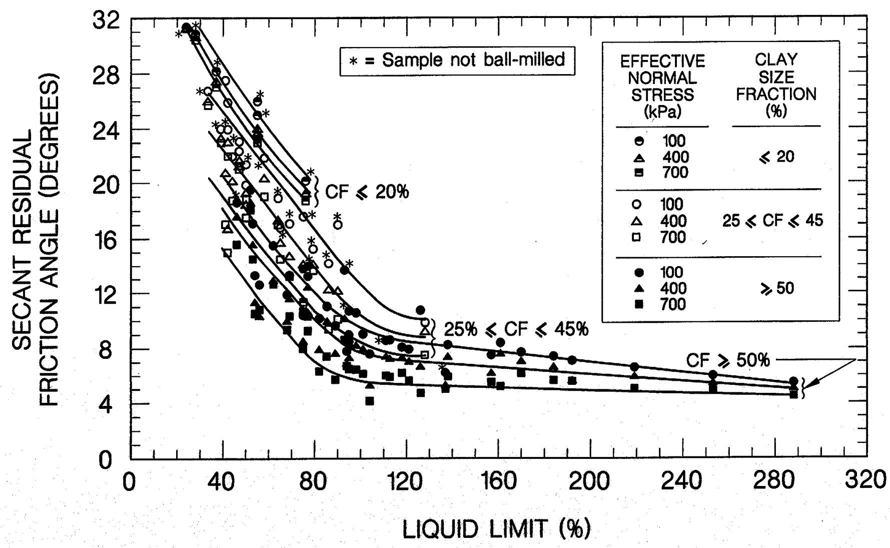


Figure 7.5c. Empirical Correlation Between Drained Residual Friction Angle of Fine-Grained Soil and Ball-Milled Liquid Limit (Stark and McCone, 2001)

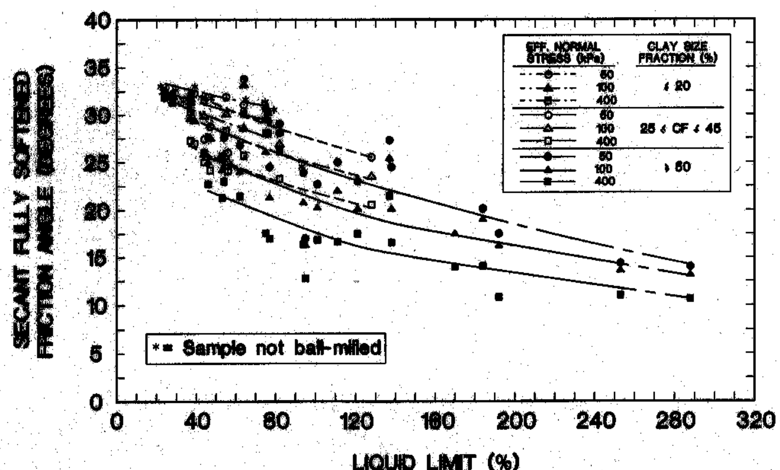


Figure 7.5d. Empirical Correlation Between Fully Softened Friction Angle of Fine-Grained Soil and Ball-Milled Liquid Limit (Stark and McCone, 2001)

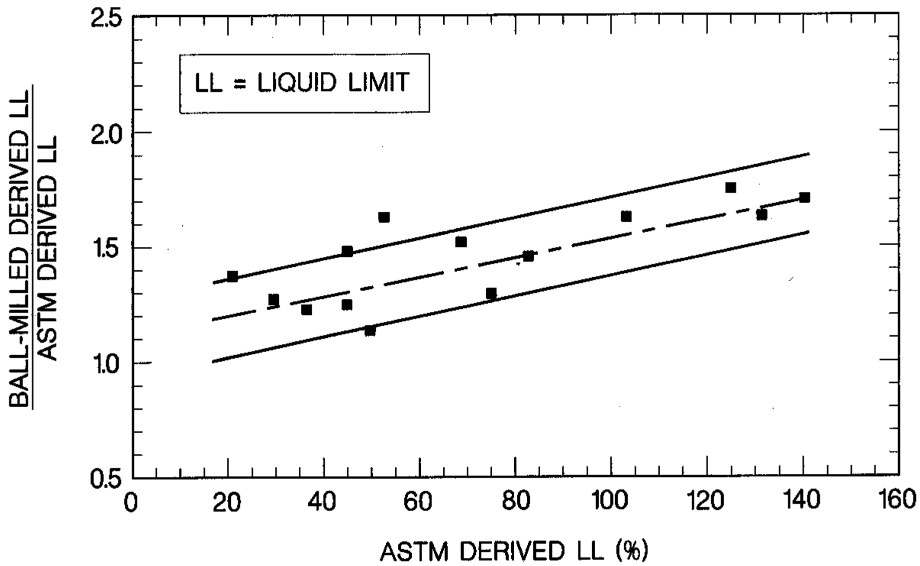


Figure 7.5e. Ratio for Ball-Milled and ASTM Values of Liquid Limit (Stark and McCone, 2001)

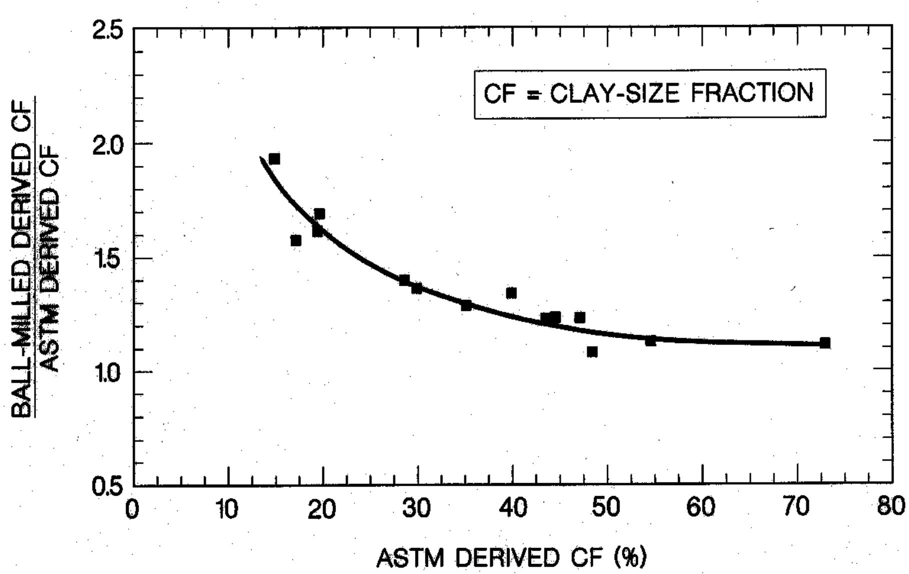


Figure 7.5f. Ratio of Ball-Milled and ASTM Values of Clay-Size Fraction (Stark and McCone, 2001)

7.3.3 Laboratory Testing

(a) General Considerations

Laboratory testing can be used to evaluate the load-deformation response and shear strength of soil samples. Laboratory equipment available for shear-strength testing includes the following:

- The triaxial compression test (TC) is a relatively common laboratory test that can be used for the evaluation of drained or undrained shear strength parameters. The applied load is measured in terms of deviatoric stresses, and deformation is measured in terms of axial strains.

- Unconfined compression tests are simply UU triaxial compression tests with zero cell pressure. Unconfined compression tests are only useful for crude estimation of total stress strength parameters, and tend to provide conservative results. These strengths can generally be applied only for an "unconsolidated" condition (i.e., no field consolidation since sample retrieval), and only for the location in the ground from which the sample was retrieved.

- The direct shear test (DS) is the most commonly used shear strength test due to its operational simplicity. In southern California, the test is often run on specimens retrieved from California samplers, which (as noted in Section 6.2) are likely to be significantly disturbed. DS test results for such specimens are very approximate. In the DS test, applied load is measured in terms of shear stress, and deformation is measured in terms of shear displacement (not strain). The ASTM procedure for this test is formulated to achieve drained shear. True undrained conditions cannot be obtained because pore pressures dissipate during shear. The direct shear test controls the location of shearing and is therefore useful for testing specific failure surfaces. DS testing devices can be used to subject a sample to multiple cycles of shearing, which allows an estimation of residual strength. Unfortunately, the results may be unconservative (Watry and Lade, 2000), and should always be checked against either correlations (Stark & McCone, 2001) or results of ring shear testing (discussed below).

- Ring shear tests can be used to estimate the residual strengths corresponding to large displacements in reconstituted (bulk) samples. Ring shear devices cannot be used with undisturbed soil specimens from the sampler types discussed in Section 6.0.

- Although mostly research tools at this point, direct simple shear and torsional shear testing provides a reliable means of evaluating either undrained or drained stress-strain response of soil.

The test procedures recommended for several classes of geologic materials are summarized in Table 7.2 below. Note that reference is made here to the general material types (and recommended drainage condition) previously described in Section 7.2.1.

Table 7.2 Recommended Shear Test Procedures
Relative to Material Type and Drainage Conditions

Material	Appropriate Drainage Condition	Recommended Test*
Sand, static loading	Drained	DS, DTC
Saturated Clayey material at low OCR, static loading	Undrained	UTC
Very Stiff Clay & Clayey Bedrock, static loading	Undrained (also check drained)	UTC (DS, DTC)
Soil or soft bedrock @ residual	Drained	DS, RS
Any soil type, unloading	Drained (check undrained if soft clay)	DS, RS (UTC)

*DS=Direct Shear (drained)
DTC=Drained Triaxial
UTC=Undrained Triaxial
RS=Ring Shear

(b) Laboratory Testing: Direct Shear Test

i. Test Procedures

The direct shear test is the most common laboratory test used in southern California to obtain strength parameters for slope stability analyses, therefore additional discussion and guidelines for its use are included below. The direct shear test should be performed in accordance with the requirements of ASTM-D-3080. The direct shear test is only useful for estimating drained strength parameters. As noted previously, true undrained conditions cannot be obtained with the direct shear test because water flow into or out of the sample is not controlled; therefore, dilation/contraction of the shear plane cannot be controlled.

The Committee is aware of a few published references that have indicated that direct shear tests, performed at rapid rates, sometimes have been used to approximate "undrained" strength parameters for cohesive soils (Lambe, 1951; Carter, 1983; Jumikis, 1984; Bowles, 1984; 1992; Liu and Evett, 1997; Budhu, 2000). Because truly undrained conditions (without volume change) cannot be achieved in normal direct shear tests and drainage conditions cannot be monitored or controlled in rapid, so-called "undrained" direct shear tests, this committee cannot endorse such practice. Furthermore, the absence of an ASTM standard for that test makes it a non-standard test that in practice will vary in procedure and quality from consultant to consultant, and one that has not benefited from a comprehensive review and comparison with truly undrained tests. Although this committee cannot endorse such a practice, some Committee members believe that the appropriate regulatory agencies have the power to decide under which testing conditions (if any) rapid, so-called "undrained" direct shear tests can be used to estimate undrained strength parameters in their individual jurisdictions. Other Committee members believe that the use of rapid deformation rates in the direct shear test device (in an effort to approximate undrained strength parameters) should not be allowed at this time, because it can lead to unreasonable and unconservative estimates of the undrained shear strength.

The following guidelines should be adhered to so that the test results can be used for slope stability analyses.

1. The dry density and moisture content prior to shear should be determined. That can be achieved by measuring the weight of the ring sample prior to testing and determining the moisture content using an adjacent ring.

2. Samples tested for static stability analyses should be saturated unless the engineer can convincingly demonstrate that saturation of the soil during the design life of the slope is unlikely. Samples tested for seismic stability analyses may be tested at field moisture conditions that are likely to exist at the time of the earthquake. For non-irrigated slopes, that may be the long-term average field moisture condition. For irrigated slopes, samples should be tested under saturated conditions. It should be noted that soaking a sample from both top and bottom can result in trapped air inside of the sample. It is often advantageous to soak samples only from the bottom until the surface of the sample suggests that soaking has achieved saturation by capillary rise.

3. Normal stresses need to be consistent with the problem being analyzed. For example, to analyze the surficial stability of a slope requires knowledge of the shear strength at normal stresses on the order of only 200 psf, which requires testing at very low confining stresses.

4. In order to obtain drained strength parameters, the speed of the direct shear test needs to be slow enough to ensure that pore pressures dissipate inside the sample. According to ASTM, the maximum speed is a function of t_{50}, which can be determined from consolidation theory using the Casagrande or Taylor methods (e.g., Holtz and Kovacs, 1981). Currently, ASTM D-3080 specifies that the time to failure is to be greater than $50 \cdot t_{50}$. Table 7.3 provides guidelines to assist in the specification of deformation rate for a direct shear test. These are based on correlations between coefficient of consolidation (c_v) and liquid limit from the U.S.

Navy Manual DM 7.01 (NAVFAC, 1986). Note that times to failure should generally not be smaller than these values (unless supported by material-specific c_v data). The recompression times are intended for use with over-consolidated materials such as bedrock (laboratory consolidation testing on these materials may not be practical, so the values in Table 7.3 can be used in lieu of material-specific tests).

5. At the end of the test, the sample should be opened to verify that the center of the sample is saturated and that no oversized fragments (per ASTM) are present inside of soil samples. In addition, the final moisture content of the sample should be determined and the degree of saturation computed.

6. The direct shear box should be periodically opened during repeat shear tests to remove accumulated soil that has squeezed out between the upper and lower halves of the shear box.

7. In accordance with ASTM, the following should be reported: Initial and final moisture contents, dry density, t_{50} (except for rock), speed of testing, stress-deformation plots, and strength plots.

The rate of testing as described in Item 4 above most significantly affects the peak strength. If a consultant does not wish to perform these slow direct shear tests to establish peak strengths, ultimate or residual strengths evaluated from a backbone curve (see below) may be used in lieu of peak strengths.

To obtain residual strength parameters or to estimate the "ultimate" strength from the inflection point on the backbone curve shown in Figure 7.4, it is necessary to repeatedly shear a sample under a constant normal load. The sample may be manually returned to its original position at the end of each cycle of shearing or the sample may be sheared in the opposite direction. The results of the multiple cycles of shearing should be plotted together to establish the ultimate strength (as illustrated schematically in Figure 7.6). The Committee consensus is that at least three cycles of loading will be required to establish the backbone curve. One cycle is not adequate because it is not practical to achieve perfect alignment of sample shear surfaces with the direction of shear in a direct shear test. Addition cycles of shearing are required to establish the residual strength. It is the consensus of the Committee that at least 5 cycles are required to estimate residual strength. The maximum deformation rate that should be used to establish these ultimate/residual strength parameters is 0.05 in/min in the initial cycles and 0.01 in/min in at least the last two cycles of a repeated direct shear test.

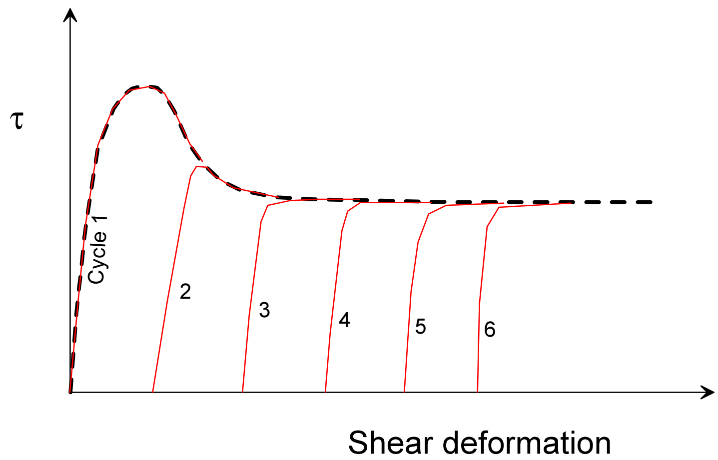


Figure 7.6. Schematic of Multiple-Cycle Direct Shear Test Results

Table 7.3. Reference Values of Time-to-Failure in Drained Direct Shear Test

Liquid Limit	Sample Condition	Time to Failure: (hrs)*
40	Over Consolidated	0.25
	Normally Consolidated	1.5
	Remolded	6.0
60	Over Consolidated	1.5
	Normally Consolidated	4.0
	Remolded	15.0
80	Over Consolidated	4.0
	Normally Consolidated	10.0
	Remolded	30.0

* assuming 1.0 inch sample height and double drainage (multiply recommended times by 4.0 if drainage is only provided on one side of sample).

ii. Remolded Samples

Direct shear testing is often performed on remolded samples to evaluate either fully softened or residual strengths. Remolded samples should be prepared to approximate either the existing or the most critical anticipated conditions. The soil moisture content and density must both be carefully selected and controlled to achieve a sample that will yield a representative shear strength. The Committee recommends that samples that will be tested with a direct shear apparatus be remolded using the following guidelines. A bulk sample of the soil should be moisture conditioned to a moisture content at or above the optimum moisture content as determined by ASTM D1557. Care should be taken not to compact fine-grained soil dry of the optimum moisture content as this may produce an unstable soil structure subject to post-peak strength loss. The prepared soil should be carefully weighed to obtain the amount required to exactly fill the direct shear ring at the desired density. The material should be compacted into the ring in three equal layers or lifts using a mallet and a solid cylindrical steel bar with an outside diameter equal to the inside diameter of the shear ring. (The surface of the preceeding layer should be roughened before placing and compacting the next layer.) It is preferable to obtain two steel bars with lips that extend beyond the ring at heights of one third and two thirds the shear ring height. This will allow the sample to be relatively uniformly compacted to a constant density.

The exception to the above remolding procedure applies to samples that are prepared to obtain a fully softened strength per the procedure in Stark and Eid (1997). Those samples should be placed as a paste directly into a ring suitable for use in the direct shear box, and allowed to consolidate under the desired normal stress prior to the onset of shear. During sample preparation, additional soil should be added as the sample consolidates to achieve the required sample height for testing.

iii. Intact Rock: Evaluation of Base Friction Angles

A conservative estimate of strengths along unweathered joint surfaces in rock masses can be obtained by direct shear testing of pre-cut rock specimens along the smooth cut surface. This will typically provide a conservative strength estimate because actual joint surfaces have asperities not present in the lab specimen. Specimens prepared in that manner typically have no significant strain softening. Alternatively, the rock may be repeatedly sheared without pre-cutting the sample. Sampling for this type of testing was discussed in Section 6.2.

(c) Triaxial Compression Test

Triaxial tests permit control of the applied principal stresses to the test sample and the drainage conditions. Because the water flow into or out of the sample can be controlled, triaxial tests can be used to measure the drained and undrained shear strength of test samples. In addition, the pore water pressure measurements can be made accurately on saturated samples; therefore, a single triaxial test on a saturated specimen can be used to determine effective and total stress shear strength parameters.

There are three types of triaxial tests: (1) consolidated undrained test (CU), in which the sample is first consolidated to a predetermined pressure and no drainage is permitted during shearing of the sample; (2) consolidated drained test (CD), in which the sample is first consolidated to a predetermined consolidation pressure and then drainage is permitted during shearing; and (3) unconsolidated undrained test (UU), in which drainage is not permitted during the application of confining pressure or shear.

As described in Table 7.2, CU or UU tests are recommended to determine the undrained shear strength of soft clay under static loading. In addition, CD tests are recommended together with the drained direct shear test to determine drained strengths of sand, very stiff clay, and clayey bedrock. The following additional discussion and guidelines are provided in this section with regard to the use of CU and CD tests for slope stability problems: CU tests should be performed in accordance with ASTM D4767-95, UU tests in accordance with ASTM D2850-95 (1999), and CD test in accordance with U.S. Army Corps of Engineers EM1110-2-1906.

In piston-type test equipment (in which the axial loads are measured outside the triaxial chamber), piston friction can have a significant effect on the indicated applied load, and measures should be taken to reduce the friction to tolerable limits.

The specimen cap and base should be constructed of lightweight material and should be of the same diameter as the test specimen in order to avoid entrapment of air at the contact faces.

The porous stones should be more pervious than the soil being tested to permit effective drainage.

Rubber membranes used to encase the specimen should provide reliable protection against leakage, yet offer minimum restraint to the specimen. Commercially available rubber membranes having thicknesses ranging from 0.0025 in. (for soft clay) to 0.01 in. (for sand or clay containing sharp particles) are generally satisfactory for sample diameters less than 2.5 inches. Rubber membranes about 0.01 in. or greater in thickness are suitable for larger specimens.

The sample specimen height-to-diameter ratio should be between 2 and 2.5. The largest particle size should be smaller than 1/6 the specimen diameter. If, after completion of a test, it is found based on visual observation that oversize particles are present, that information needs to be included in the report.

The average height of the specimen should be determined from at least four measurements, while the average diameter should be determined from measurements at the top, center, and bottom of the specimen as follows:

$$D_{avg} = \frac{D_{top} + 2D_{center} + D_{bottom}}{4} \qquad (7.2)$$

The confining pressures for CD triaxial tests should be chosen such that the confining pressures approximately simulate the possible state of effective stresses (before and after the construction) at the depth at which the sample was obtained. Usually, three confining pressures are chosen. The selected confining pressures can be equal to the existing effective overburden pressure (i.e., vertical effective stress), the maximum or minimum anticipated future vertical pressure, and an intermediate pressure. For CU testing, specimens can be consolidated to stresses beyond their preconsolidation pressure to minimize sample disturbance effects, and then unloaded to various overconsolidation ratios as needed for the site under consideration. As described in Section 7.2, the laboratory shear strengths are then normalized by the major principal consolidation stress to define OCR-dependent normalized strength parameters. These normalized parameters are combined with the major principal effective stresses in situ to estimate in situ shear strengths (Ladd and Foott, 1974).

Before calculating the deviator stress, the area of the specimen should be corrected for the corresponding axial strain.

When saturation of triaxial specimens is required to simulate field conditions, it is common practice to use back-pressure to achieve saturation. Using high back-pressures to achieve full saturation should be avoided as it might produce higher shear strengths than can be expected in the field. To avoid excessively high back-pressures, it is recommended that a high degree of saturation (> 80 percent) be achieved by first percolating de-aired water under a small hydraulic gradient through the specimen until air stops bubbling from it.

Multistage triaxial tests are usually not recommended because of the likelihood of overstraining specimens and thereby significant errors in the assessment of shear strengths.

Filter paper side drains, often used to accelerate consolidation, should not be routinely used in triaxial tests because they may lead to errors in strength measurement.'

In coarse soil, corrections for the effect of membrane penetration should be applied according to established standards of practice (ASTM D4767-95/ASTM D2850-95).

The strain rate for CD tests with pore pressure measurements should be such that the pore water pressure fluctuation is negligible (pore pressure fluctuations should not be greater than 5 percent of the effective confining pressure).

The mode of failure of a triaxial sample should be observed and recorded in the report, and if the sample fails on a distinct plane, the angle of inclination of the plane of failure should be measured and recorded in the report. Failure is typically defined for CD tests as the maximum deviatoric stress, $(\sigma_1'-\sigma_3')_f$.

For CU tests, failure can be defined either as the maximum deviator stress $(\sigma_1'-\sigma_3')_f$, the maximum obliquity, $(\sigma_1'/\sigma_3')_f$, or the stress at a certain specified axial strain. For dilative samples, a maximum deviator stress criteria may not be determined as its value will continue to increase with deformation. However, maximum obliquity value will reach a maximum and will not increase with the deformation. Therefore, for contractive samples, maximum obliquity criteria should be used for defining the failure. For dilative samples, either maximum deviator stress or maximum obliquity criteria will provide the same measure of shear strength; however, typically the maximum deviator stress is used in slope stability

(d) Laboratory Test Data Interpretation

The number of tests needed to estimate the shear strength of a geologic unit depends on factors such as local experience with the material, continuity of strata, spatial variability of properties, and consequences of erroneous estimation. When the number of tests performed is limited, appropriate conservatism should be used to select shear-strength values for slope stability analysis. The following general guidelines should be considered when testing shear-strength samples, and analyzing and applying their results.

If data are being developed to estimate the shear strength of a relatively homogeneous deposit (such as a uniform natural deposit or an artificial fill), a sufficient number of tests should be performed to characterize the variation that is likely to result from the natural process or construction techniques, considering the materials that are available to form the deposit. The results from a number of tests can be averaged, provided they are weighted in proportion to their abundance in the slope being analyzed. Alternatively, each layer could be entered into the slope stability analysis. If a wide variation in shear strength is observed across a large project site, it is necessary to verify that the strengths used for analysis of a specific slope are representative of the materials at that location.

If data are being developed to estimate the across-bedding strength of a layered deposit, the tests should be performed on representative material samples from each of the types of layers present. In many cases, an approximately weighted average value of shear strength can be used to model the across-bedding strength. Summary plots of shear strength data for each type of material in the layered deposit should be prepared. The test results from each type of material in a layered deposit should be averaged first. Then those averaged results should be weighted in proportion to their abundance and combined with similar results from other layers to obtain an overall weighted average. The engineer should be sure to consider the possibility that large-scale properties such as variations in cementation and fracturing could affect the strength of the deposit in a manner that might not be adequately represented by the laboratory test results.

If data are being developed to estimate the along-bedding strength of a layered deposit, it is important that the testing programs be conducted to effectively characterize the layers with the lowest strengths. Index tests, classification tests, or some other acceptable means should be used to select the weakest layers for testing. The results from tests on the different samples should be summarized on composite shear-strength plots and a value no greater than the lower bound of those data should be selected for use in stability analysis, unless it can be adequately demonstrated that the type of material that produced the lower-bound is not present at a specific location being analyzed. It is important to recognize that a typical laboratory-testing program may not be sufficient to find the layer with the lowest strength at a particular site. Consequently, past local experience and local presumptive strengths, should be considered when deciding whether the lowest along-bedding strength obtained from a laboratory-testing program is sufficiently low enough for use in a slope stability analysis.

Another important consideration for layered deposits is the strain compatibility of shear strengths. Engineers should select strengths for each layer that are consistent with the level of strain developed at different stages of loading.

7.3.4 Field Testing

In-situ vane shear testing (ASTM 2573) is a reliable means of evaluating the undrained strength of cohesive soil. The test consists of inserting a metal vane into the soil and rotating it until failure is reached in the soil adjacent to the vane.

The test is best suited for very soft to stiff clay and it should be avoided in very stiff and/or fissured clay. Unreliable readings may result when the vane encounters sand layers, stones, or if the vane is rotated too rapidly.

The undrained shear strength can be calculated based on the amount of torque at failure and the vane dimensions. For a typical field vane, with a 2:1 height:diameter ratio, Chandler (1988) proposed the following relationship for soil with a moderate plasticity:

$$S_u = \frac{0.86 \cdot T}{\pi d^3} \tag{7.3}$$

The vane shear test can overestimate the undrained shear strength of a soil, especially in materials with a high plasticity index. The reason for this overestimation is creep strength loss effects, and to a lesser extent, anisotropy effects. To account for these effects, a correction factor, μ, is often used to relate the field shear strength to the measured strength:

$$(S_u)_{field} = \mu (S_u)_{vane} \tag{7.4}$$

The relation between the correction factor, μ, and the plasticity index, PI, has been obtained from field case history data and is shown in Figure 7.7.

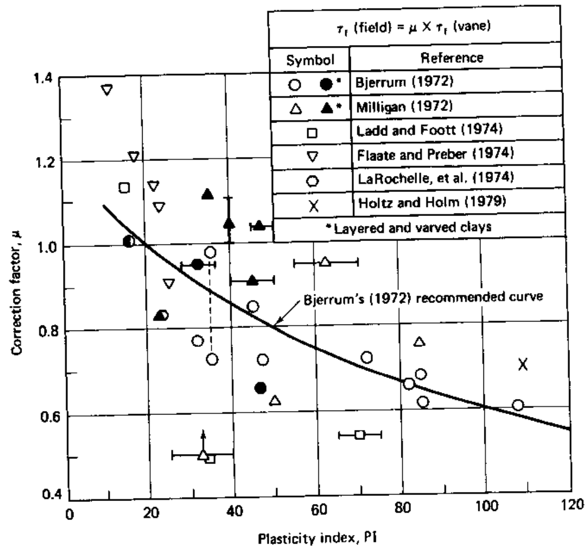


Fig. 7.7. Correlation Factor for the Field Vane Test as a Function of PI, Based on Embankment Failures (from Holtz and Kovacs, 1981)

7.3.5 Back Calculation of Strength Along a Failure Surface

Existing landslides offer the opportunity to estimate the average shear strength properties along the failure surface by mathematical methods. This procedure is generally referred to as back calculation or back analysis. The procedure requires the determination of the configuration of the landslide failure surface relative to the topography at the time of failure, variability in earth materials along the failure surface, the subsurface water level at the time of failure, external loading conditions, and the appropriate soil density. Once the above information is known, a mathematical analysis method appropriate to the slide configuration is chosen. The data described above are input into the analysis method, and an initial estimate is made of the shear strengths along the failure surface. The shear strength parameters are then adjusted and the analysis repeated until a factor of safety of 1.0 (FS=1.0) is obtained. This method provides different sets of cohesion, *c*, and friction angle, ϕ, which satisfy FS = 1.0. The engineer then selects an appropriate combination of *c* and ϕ. These strength parameters can then be utilized in the evaluation of alternate repair procedures. Skempton (1985) compared drained shear strengths obtained by careful testing of high-quality slip-surface samples with strengths determined by back calculation of the slides and found good correlation, indicating that the back-calculation method is valid for drained failures.

The back-calculation method of determining the shear strength of earth materials is often a better method to utilize (when applicable) than laboratory testing because this method eliminates problems associated with sample size and number, as well as inherent problems with different shear test apparatus. The method essentially utilizes nature to perform an in-situ shear test. The back-calculation method does require a general knowledge of material characteristics and drainage conditions at the time of failure to allow an initial determination of the probable ranges of the angles of internal friction for the earth materials along the failure surface. In addition, a general assumption of the relative contribution of the cohesion intercept and angle of internal friction to the soil shear strength must be made. Often, an estimate of the friction angle can be obtained through local knowledge or review of published shear strength values for similar materials. Laboratory testing on a limited number of samples also can be utilized to establish the range of anticipated soil shear strength parameters to use when performing the back analyses.

Although the usefulness of back calculation or back analysis for the determination of shear strength parameters is beyond debate, there are several problems that must be recognized. The first problem is the time frame of occurrence of the landslide or failure. Landslides that develop as a result of a construction excavation or creation of a cut slope may not be suitable for use in determination of the soil shear strength due to the relatively rapid unloading that led to the landslide. The soil along the failure surface may not have uniformly reached their shear strength at the time of failure, especially if the failure occurs in a progressive fashion. This may result in the overestimate of the strength parameters or, stated differently, the factor of safety of the slope during the failure might have been less than 1.0.

Another situation where back analysis may not be applicable is the determination of shear strength parameters for an ancient (geologically speaking) landslide. Ancient landslides likely occurred under climatic conditions different than those existing today. Moreover, the surface topography of the slide may have been significantly altered by erosion or deposition since its initial failure. Finally, ageing of soil along the slip surface could have increased material strengths since the slope failure. Therefore, neither the topographic conditions, subsurface water conditions at the time of failure, nor shear strength conditions at present, are accurately known. If reliable estimates of the topographic conditions at the time of failure and the subsurface water levels cannot be made, the shear strengths obtained from back analyses can only be used to estimate a range based on appropriate variations in the unknown parameters.

Care should be taken when back-analyzing landslides that have moved a significant distance because the strength parameters determined using the groundwater and topographic conditions at the end of the failure movement will often be different and generally less than strength parameters determined using conditions just prior to the initiation of slide movement.

8 SOIL UNIT WEIGHT

The soil unit weight is required for the analysis of slope stability. The added weight due to the presence of subsurface water is accounted for by using the saturated unit weight of the soil. The use of the saturated unit weight (γ_{sat}) of the soil is conservative for most analyses. Although variations in moisture content (varying from dry to saturated) are possible, slope stability analyses should be performed using the saturated unit weight (unless specific justification for doing otherwise is provided by the consultant and approved by the regulatory reviewer). The estimation of saturated soil unit weight can be evaluated from the dry unit weight (γ_d) as follows,

$$\gamma_{sat} = \gamma_w + \gamma_d \left(\frac{G_s - 1}{G_s} \right) \tag{8.1}$$

where G_s = specific gravity of solids (typically 2.65-2.75),
γ_w = unit weight of water (62.4 pcf for fresh water)

In addition, relatively small (5 to 10 pcf) changes in density typically have little influence on the results of slope stability analyses. Saturated unit weights should be obtained from laboratory moisture-density tests on driven samples or conservative estimates from published sources such as the Slope Stability Reference Guide for National Forests in the United States (Hall et al., 1994).

9 STATIC SLOPE STABILITY ANALYSIS

Slope stability analyses involve a comparison of the gravity induced stresses in a slope to the available soil strength and any externally provided resistance (e.g., retaining walls). Available static equilibrium methods solve for one or more of the three equations of equilibrium: horizontal force, vertical force, and moment. The availability and speed of personal computers has made the use of methods of analysis that satisfy all equations of equilibrium feasible for practicing engineers.

Proper analysis of the static stability of a slope requires representations of the slope configuration, external loading conditions, distribution of earth materials, subsurface water conditions, material densities, and material strengths. The specification of those input parameters has been covered previously in Chapters 4-7.

9.1 FACTOR OF SAFETY

Static limit equilibrium stability analysis methods calculate the factor of safety by satisfying one or more of the three equations of static equilibrium: horizontal and vertical force equilibrium, and moment equilibrium. The factor of safety (FS) is defined as,

$$FS = \frac{Available\ Soil\ Shear\ Strength}{Equilibrium\ Shear\ Stress} \tag{9.1}$$

The slope is considered to be at the point of failure when the factor of safety equals one or the available soil shear strength exactly balances the shear stress induced by gravity. A slope has reserve strength when FS > 1.

Generally, the probability of failure decreases as the factor of safety increases. However, a unique relationship between probability of failure and FS cannot be established because of the wide variability in uncertainties in input parameters from site-to-site. In most cases, the most pronounced sources of uncertainty in a slope stability analysis are the soil strength and groundwater conditions. Other factors contributing uncertainty include the imperfect nature of mathematical models for slope stability calculations and the ability of the analyst to find the critical failure surface geometry.

Historically, the most commonly required factors of safety in southern California have been 1.5 for static long-term slope stability and 1.25 for static short-term (during construction) stability. Those factors of safety were established when computations were performed with slide-rules, when analysis methods solved at best two conditions of equilibrium, when only a few potential failure surfaces were analyzed, and when our understanding of factors influencing the shear strength of soil was less advanced. The level of uncertainty associated with those analyses justified the use of relatively high factors of safety.

The availability and speed of personal computers has allowed the development of more precise methods of analysis, which satisfy all three equations of static equilibrium, and the analysis of hundreds to thousands of potential failure surfaces. Therefore, the uncertainty related to computational methods and determination of the critical failure surface has been significantly reduced in recent years. Accurate representation of the soil shear strength for the problem being solved therefore introduces the highest level of uncertainty into current analyses. The Committee believes that the current static factors of safety remain applicable in cases where the shear strength of soil is determined by limited laboratory testing or by the use of the median values from standard correlations. However, we also believe that consideration should be given in the future to the use of lower factors of safety when uncertainty related to the shear strength is relatively small. For example, uncertainty is reduced when the shear strength is determined by back analysis of a well documented slope failure (in terms of geometry and water conditions). The Committee is not prepared to recommend specific lower safety factors at this time, but believes that this topic deserves consideration by controlling agencies.

The use of a factor of safety greater than 1.5 for static analyses is recommended if a slope in fractured or jointed cemented bedrock is analyzed using peak strength parameters derived from high quality samples of unfractured material. The use of a higher factor of safety is suggested in this instance because the joints and fractures introduce random planes of weakness into the deposit, which can significantly reduce the overall shear strength of the deposit. It is the Committee's judgment that factors of safety as high as 2.0 should be considered when a cemented material exhibits significant post-peak strength loss and contains a significant number of fractures in the location being analyzed. It should be noted that this higher factor of safety is not intended to be used when shear strengths are evaluated from de-aggregated samples.

9.2 METHODS OF STATIC SLOPE STABILITY ANALYSIS

The static stability of slopes is usually analyzed by dividing a profile view of the soil into a series of slices and calculating the average factor of safety for all of those slices using a limit equilibrium method. For simple profiles composed of a single homogeneous earth material, the factor of safety is also sometimes calculated by analyzing the stability of the entire profile using some simplifying assumptions, as in Taylor's friction circle method. Those analyses require knowledge of the slope geometry and estimates of soil strength. Limit equilibrium methods assume that the soil acts as a rigid mass and do not require information about its stress-strain behavior. An inherent assumption in the use of the limit equilibrium methods is that the factor of safety is the same for all of the slices and the shearing resistance is mobilized simultaneously along the entire failure surface. Because many failures mobilize progressively, that may not be a valid assumption for all slopes. However, in spite of those limitations, the method is in widespread use and experience gained from its application throughout California suggests that slopes can be safely designed using that analytical procedure.

9.2.1 Available Limit Equilibrium Methods of Analysis

Many limit-equilibrium methods of slope stability analysis are available (Table 9.1). Historically, the simpler methods were developed before the age of computers and the more complex methods followed after. The various methods of limit equilibrium analysis differ from each other in two regards: 1) different methods use different assumptions to make up the balance between the number of equations of equilibrium and the number of unknowns and 2) different methods use different assumptions regarding the location and orientation of the internal forces between the assumed slices. Some analysis methods do not satisfy all conditions of equilibrium or even the conditions of force equilibrium. A summary of some of the commonly used methods is provided in Table 9.1.

9.2.2 Accuracy

The accuracy of the available limit equilibrium slope stability methods can be compared by examining:

1. Their inherent ability to handle the mechanics of slope stability, and

2. The limitations on accuracy that result from there being too few equations of equilibrium to calculate the factor of safety without excessive use of assumptions.

Computational accuracy considers only the computation of shear stress required for equilibrium and the normal stress. Therefore, computational accuracy is not the same as the accuracy of the analysis as a whole, which is most significantly influenced by the uncertainty in input parameters (such as soil strength). However, in situations where good quality sampling and testing have revealed consistent strength parameters or where regional knowledge dictates the use of specific parameters, the method of analysis can significantly affect the calculated FS.

The methods of Morgenstern and Price, Spencer, Sarma, Taylor, and Janbu's generalized procedure of slices satisfy all conditions of equilibrium and involve reasonable assumptions. Bishop's modified method does not satisfy all conditions of equilibrium, but is as accurate as methods that do, provided it is used only for circular surfaces. Duncan (1996) has found all of these methods to provide answers within 5% of each other.

Table 9.1. Characteristics of Commonly Used Methods of Limit Equilibrium Analysis (after Duncan, 1996)

Method	Date	Equilibrium Conditions Satisfied	Shape of Slip Surface	Assumptions
Friction Circle Method (Taylor)	1937	Moment and force Equilibrium	Circular	Resultant tangent to friction circle
Ordinary Method of Slices (Fellenius)	1927	Moment Equilibrium of entire mass	Circular	Normal force on base of slice is W cos α and shear force is W sin α
Method of Slices (Fellenius)	1910	Force equilibrium of each slice		No interslice forces
Bishop's Modified Method	1955	Vertical equilibrium and overall moment equilibrium	Circular	Side forces are horizontal
Janbu's Simplified	1968	Force equilibrium	Any shape	Side forces are horizontal
Modified Swedish Method (U.S. Army Corps of Engineers Method)	1970	Force equilibrium	Any shape	Side force inclinations are equal to the parallel to the slope
Lowe and Karafiath's Method	1960	Vertical and horizontal force equilibrium	Any shape	Side force inclinations are average of slope surface and slip surface (varies from slice to slice)
Janbu's Generalized Method	1968	All conditions of equilibrium	Any shape	Assumes heights of side forces above the base vary from slice to slice
Spencer's Method	1967	All conditions of equilibrium	Any shape	Inclinations of side forces are the same for every slice; side force inclination is calculated in the process of the solution
Morgenstern and Price's Method	1965	All conditions of equilibrium	Any shape	Inclinations of side forces follow a prescribed pattern; side forces can vary from slice to slice
Sarma's Method	1973	All conditions of equilibrium	Any shape	Magnitudes of vertical side forces follow prescribed patterns

9.2.3 Acceptable Methods for Slope Stability Analyses

Considering the foregoing statements regarding accuracy, the methods of Morgenstern and Price, Spencer, Sarma, and Janbu's generalized procedure of slices probably will yield reasonable estimates of the factor of safety for failure surfaces of any shape. However, because of the difficulty associated with selecting an appropriate force function for use with the Morgenstern and Price, and Sarma methods, and the frequent numerical instability problems associated with Janbu's generalized procedure, those methods may not be suitable for general engineering practice. As a result, the Committee recommends that Spencer's method be used for analyses of failure surfaces of any shape. In addition, we also recommend that the Taylor and Bishop modified methods be allowed for the analysis of circular failure surfaces. If a stability analysis has been performed using a method other than the Spencer, Taylor, or Bishop methods, it is recommended that the factors of safety for critical surfaces be checked using one of those three methods.

9.3 FAILURE SURFACE GEOMETRY

9.3.1 Type of Failure Surface

The failure surfaces to be analyzed for slope stability should consist of the combination of lines and circles that results in the lowest factor of safety. The slope analyzed should consider the full slope height unrestricted by property line locations. Examples of the types of failure surfaces that should be considered are illustrated in Figure 9.1 and described below. However, Figure 9.1 does not provide an exhaustive list of all possible failure modes.

- Circular failure surfaces can be used in slopes with laterally supported bedding or for slopes composed of relatively homogeneous materials such as fill slopes (Figs. 9.1 a-b).

- Failure surfaces consisting of a combination of lines that follow the weak materials (such as bedding planes, landslide slip-surfaces, faults, or joints), should be used for heterogeneous slopes with weak layers or geologic discontinuities (Fig. 9.1c).

- Potential failures along unsupported bedding planes should be analyzed when present (Fig 9.1 d).

- Composite failure surfaces that consist of linear slip-surfaces along bedding planes in the upper portions of the slope in combination with slip surfaces across bedding planes in the lower portions of the slope should be used where bedding planes form a dip-slope or near dip-slope. It may be necessary to vary the orientations of the portions of the failure surface that cross layer boundaries to create kinematically acceptable failure geometries (e.g., Figs.

9.1e-f). In general, failure geometries with a near 90-degree angle in the lower portion of the slope should be avoided as these geometries will lead to unreasonable high normal stress concentrations near the right angle bend in the failure surface.

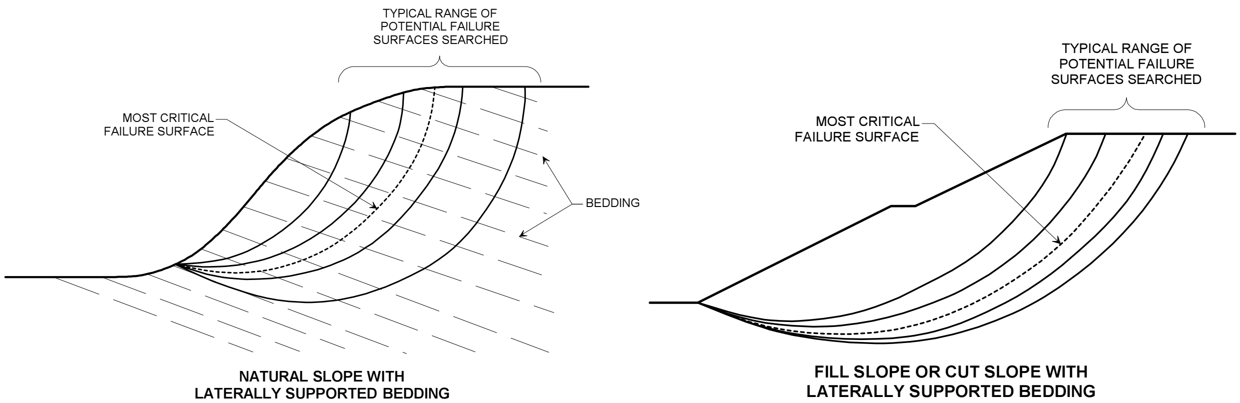


Figure 9.1a - b. Examples of Use of Circular Failure Surface Geometry

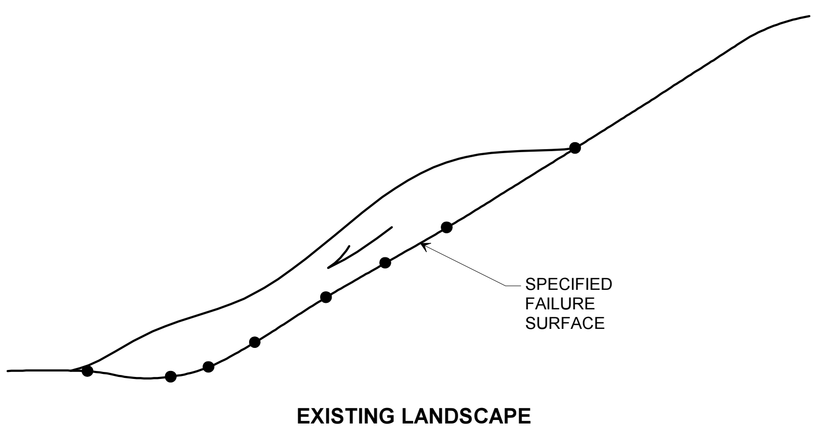


Figure 9.1c. Example of Use of Specified Failure Surface Geometry for Existing Landslide

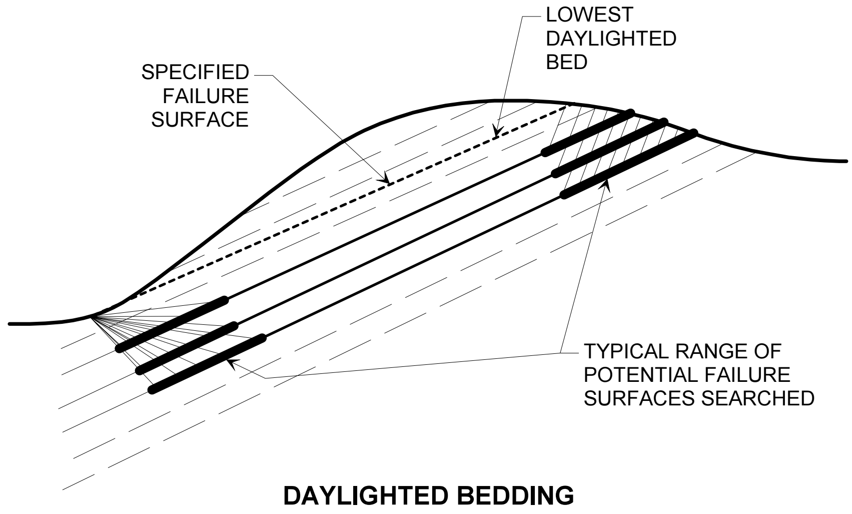


Figure 9.1d. Potentially Critical Failure Surfaces for Slope with Daylighted Bedding Planes

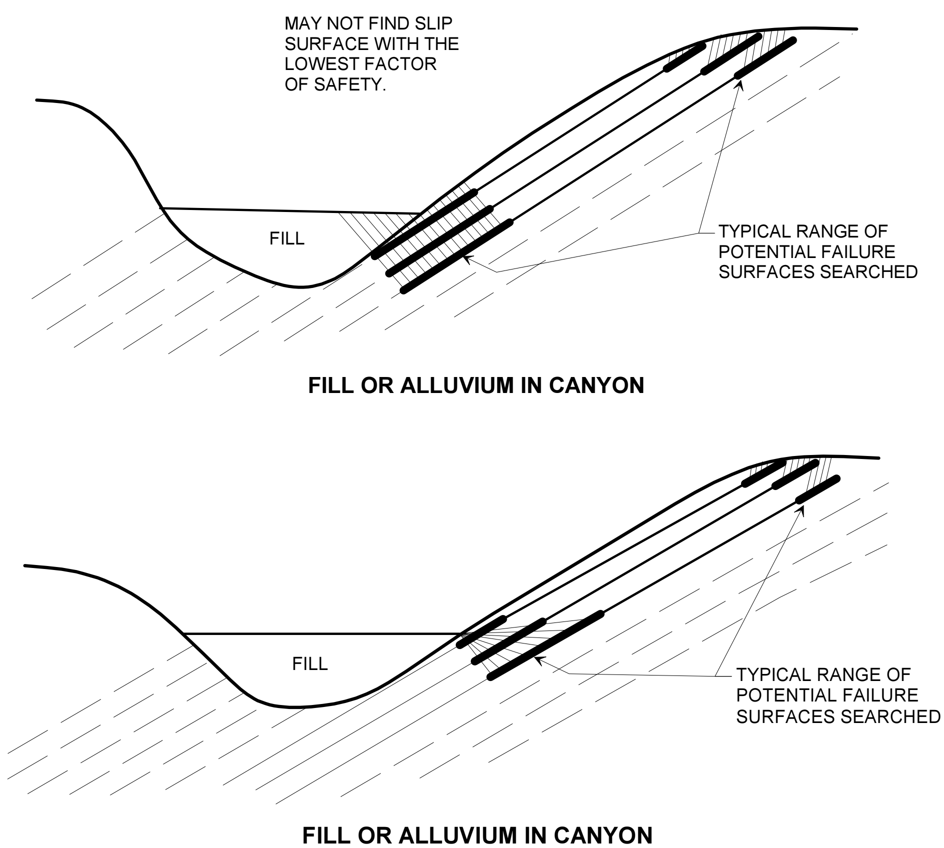
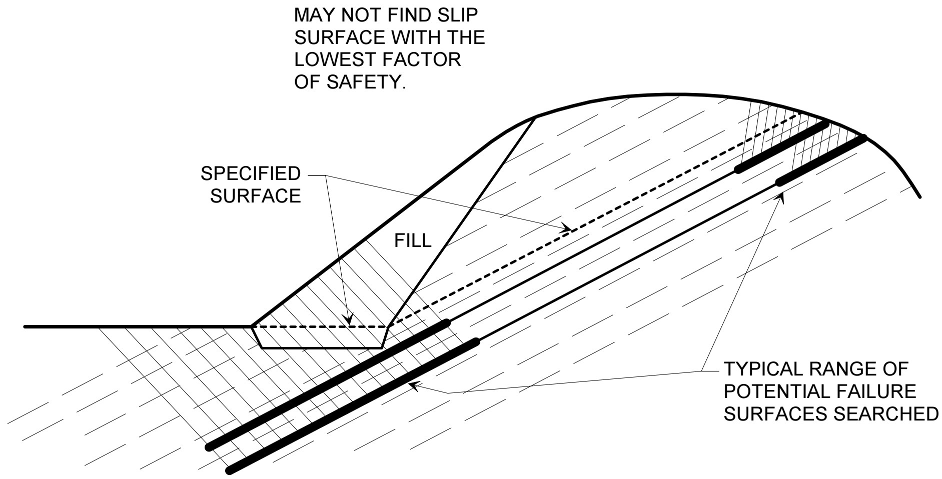
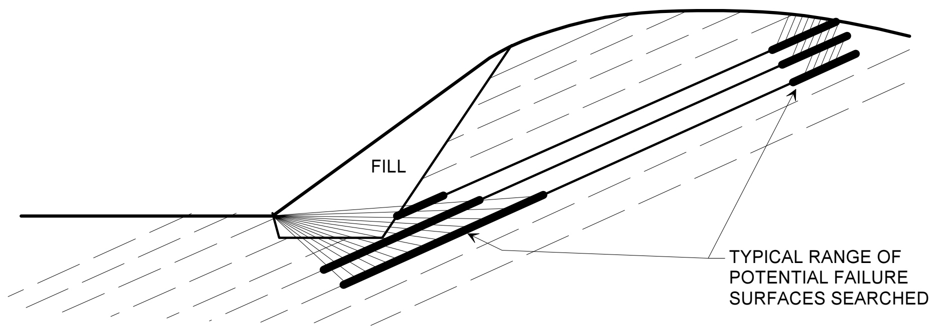


Figure 9.1e. Failure Surfaces Combining Along-Bedding and Cross-Bedding Failure - Fill or Alluvium in Canyon (bottom diagram indicates correct geometries)



BUTTRESS FILL



BUTTRESS FILL

Figure 9.1f. Failure Surfaces Combining Along-Bedding and Cross-Bedding Failure - Buttress Fill (bottom diagram indicates correct geometries)

9.3.2 Tension Cracks

Tension cracks or vertical fractures may form at the crest of a slope or near the head of a landslide as failure is approached. Tension cracks should be considered in slope stability calculations, and in some cases those cracks should be assumed to have water in them. The tension crack lateral location along the slope should be the one that produces the lowest factor of safety, but in practice it may not be necessary to expend the iterative effort needed to determine the most critical position.

For most situations, the approximate depth of the tension crack can be estimated from the following equations. If the material through which the crack will form is generally homogeneous and isotropic, the depth of the tension crack may be estimated from:

$$H_c = \frac{2c_m}{\gamma} \cdot \tan\left(45 + \frac{\phi_m}{2}\right) \tag{9.2}$$

Where H_c is the crack depth, γ is unit weight, and c_m and ϕ_m are the mobilized soil strength parameters,

$$\begin{aligned} c_m &= \frac{c}{FS} \\ \tan(\phi_m) &= \frac{\tan(\phi_m)}{FS} \end{aligned} \tag{9.3}$$

If the slope consists of laterally unsupported, interbedded weak- and strong-layers, the following formula can be used to help estimate the depth of the tension crack (from City of Los Angeles):

$$H_c = \frac{c}{FS \cdot \gamma \cdot \sin\alpha \cdot \cos\alpha - \gamma \cos^2 \alpha \tan\phi} \tag{9.4}$$

Where α = inclination of slip plane in the area where a tension crack might form, and FS = factor of safety (1.5).

The appearance of a negative shear stress along failure surfaces near the top of a failure during a stability analysis generally indicates the need for a tension crack or a deeper tension crack.

If the computer program being used does not allow for the automatic specification of a tension crack, one can be artificially input by the specification of low shear strength near the ground surface.

9.3.3 Search for Critical Failure Surfaces

No matter which method of analysis is used, it is essential to perform a thorough search for the critical slip surface to be sure that the minimum factor of safety is calculated for a slope. The searching method needs to be varied depending on the geologic conditions believed to exist in the slope.

If circular failure surfaces are to be used, a sufficient number should be generated so that a range of reasonable failure paths is considered. Care should be taken to include obvious failure initiation points such as the toe of the slope or points where the slope angle changes significantly. If the computer program used allows the user to specify a range of failure circle or surface initiation points, care should be taken to specify the range and spacing such that obvious initiation points are expressly checked by the search. If the computer program used to search for a critical circular failure surface uses a grid of centers, efforts need to be made to ensure that all local minimums are found. If the computer program works by generating a large number of circular surfaces in a random manner, the engineer needs to direct the computer to search enough surfaces so that adding more surfaces does not result in a significantly lower factor of safety.

If non-circular failure surfaces are to be used, geologic judgment and kinematics need to be considered. For example, if Spencer's method is used to generate a failure surface that has a nearly right-angle bend (see Figure 9.1e-f, upper frames) a kinematically unreasonable geometry results and the calculated factor of safety may be too high. That problem can be detected by checking for very high base-of-slice normal-stresses and shear resistances in narrow slices. Those high stresses and resistances result from the concentration of high side forces at the right-angle bend, which creates high base-of-slice normal-forces and unreasonably high shear-resistance. Spencer's analysis can yield factors of safety that are significantly higher than those produced by a simplified Janbu analysis when kinematically unreasonable surfaces are specified (dip-slope analyses with passive toe wedges can create that problem). The problem can often be resolved by searching for similar, but kinematically more reasonable surfaces, in nearly the same area (see Figure 9.1e-f, lower frames). If a computer program is used to generate a large number of non-circular randomly shaped surfaces, the engineer should carefully evaluate the results for convergence, since good geotechnical and geologic judgment can often result in finding more critical failure surfaces. To provide some guidance, several examples of procedures that can be used to search for the critical failure surface are shown on Figure 9.1

9.3.4 Search for Critical Failure Direction

Existing or potential failures that do not occur directly downslope require consideration of the critical direction of analysis (cross section direction that results in the lowest factor of safety). Landslides that do not occur directly downslope and slopes where the direction of bedding dip is oblique to the slope require that consideration be given to the direction of failure. In general, the analyst can start the search for a critical failure direction by evaluating cross sections that extend directly downslope and directly down the dip of the failure surface or bedding plane and then expanding that search to include intermediate directions, if such appear to be more critical.

9.4 GROUNDWATER CONDITIONS

Engineers performing computer-aided slope stability analyses should determine how the specific program they are using accounts for pore-water pressure and be sure that they specify it correctly. For example, in the computer program XSTABL, when a phreatic surface is used to describe pore-water pressures and that phreatic surface is above the ground, a water surcharge is applied to the ground surface. However, when a piezometric surface is used in XSTABL and that surface is above the ground, no water surcharge is applied to the ground surface. Also, when specifying a phreatic surface in XSTABL, the program assumes that equipotential lines are perpendicular to the phreatic surface to calculate the pore-water pressure head. However, if a piezometric surface is used, the pore-water pressure acting on a slide is assumed to be the vertical (elevational) head (measured from the base of the slice to the piezometric surface).

Changes in saturation of a soil may have some effect on the cohesion component of the soil shear strength by eliminating capillary tension or "apparent cohesion" in an otherwise cohesionless soil or by the reduction of the "dry strength" of a cohesive soil. The positive effects of capillary forces can easily be lost, and the use of saturated shear strength parameters is recommended. Saturation also affects the frictional shear strength because of the buoyant reduction of the normal force component caused by the pore-water pressure.

Water should be assumed within the tension crack where geologic evidence exists that tension cracks have developed near the top of similar slopes in similar earth materials.

9.5 SLOPE STABILITY ANALYSIS USING FINITE ELEMENT/ FINITE DIFFERENCE METHODS

Finite element/finite difference (FE/FD) elasto-plastic analysis of geotechnical problems has been used for large-scale and research projects, but has seen limited use for routine applications. However, the use of FE/FD procedures is required for assessments of static slope displacement, and may be desirable for stability calculations, if a complex subsurface stratigraphy is encountered (because those conditions can make it difficult to estimate the geometry of the most critical failure surface through a slope). Unfortunately, those benefits are often offset by difficulties in defining parameters for appropriate material constitutive models. Comparative studies between FE/FD and limit-equilibrium methods have shown that similar results can be obtained by each (Griffiths, 1980; 1989; Potts et al., 1990; Matsui & San, 1992; Griffiths and Lane, 1999).

Some of the advantages of the FE/FD approach to slope stability analysis over typical limit equilibrium methods include:

- No prior assumption needs to be made about the shape or location of the failure surface. Failure occurs through the zones within the soil mass where the shear strength is unable to sustain the applied shear stresses.

- Because FE/FD methods do not utilize "slices" there is no need to make simplifying assumptions about slice side forces. FE/FD methods preserve global equilibrium until failure occurs.

- If realistic soil compressibility data are available, FE/FD methods can give general information about deformations at working-stress levels.

- FE/FD methods illustrate progressive failure up to and including overall shear failure. By contouring shear strains in the zones, it is possible to highlight failure surfaces.

For non-linear analyses using complex constitutive models that attempt to reproduce volumetric changes accurately in undrained or partially drained conditions, the incremental application of gravity can produce different results than would be obtained if gravity is applied all at once. However, if a simplified elasto-plastic model is used in FE/FD analyses, the factor of safety appears unaffected (Griffiths and Lane, 1999). Therefore, if the primary goal of the FE/FD analysis is to obtain a factor of safety, a simplified Mohr-Coulomb elasto-plastic model can be used with an instantaneous gravity "turn-on" procedure (Griffiths and Lane, 1999). To determine the factor of safety (FS) from FE/FD analyses, the "shear strength reduction technique" can be used (Matsui and San, 1992). In that procedure, the FS of a soil slope is defined as the number by which the original shear strength parameters must be divided in order to bring the slope to the point of failure (as indicated by numerical non-convergence or excessive displacement). The "factored" shear strength parameters c'_f and ϕ'_f, are given by:

$$c'_f = c' / FS$$

$$\phi'_f = \arctan(\tan\phi' / FS)$$

The method would allow a different FS to be specified for the c' and tan ϕ' terms, but typically the same factor is applied to both terms. To find the slope's factor of safety, a systematic search is conducted to find the FS that initiates failure by solving the problem repeatedly using a sequence of user-specified FS values.

Modern FE/FD programs have enhanced graphical output capabilities that allow better understanding of the mechanisms of failure and simplify the output from reams of paper to useable graphs and plots of displacement. However, what remains is the concern that powerful tools such as the FE/FD method require considerable experience to properly evaluate the results.

The FE/FD method is a powerful tool which provides significant insight into the potential slope performance to the experienced user. A user should be thoroughly familiar with both the mathematical mode and the required input parameters before using this method.

9.6 SURFICIAL SLOPE STABILITY

Natural slopes and manufactured fill slopes can be subject to shallow surficial failure referred to as soil slumps or soil slips during periods of intense rainfall or excessive irrigation. These failures are typically less than about 4 feet in depth and have small thickness to length ratios. These failures are typically analyzed using the infinite slope model suggested by Campbell (1975). The infinite slope formula presented in Campbell (1975) was first derived by Skempton and DeLory (1957) based on the earlier work of Haefeli (1948) and Taylor (1948). That model assumes an infinitely long failure parallel to the ground surface with a perched groundwater surface parallel to and coincident with the ground surface. The equation for factor of safety based on that model is:

$$F = \frac{c' + (\gamma - m\gamma_w)z\ \cos^2 B \tan\phi'}{\gamma z \sin B \cos B} \tag{9.5}$$

where F is the factor of safety, c' is the cohesion intercept, z is the vertical depth of the slip surface, B is the slope angle, γ is the unit weight (density) of the soil, γ_w is the unit weight of the water, m is the fraction of z such that mz is the vertical height of the groundwater table above the slip surface, and ϕ' is the angle of shearing resistance.

The shear strength parameters applicable for use in this equation must be determined at very low normal stress (100 to 300 pounds per square foot). Direct shear tests performed at those low normal stresses can be unreliable. Therefore, it is the Committee's recommendation that tests be performed at relatively low normal stresses such as 400, 800, and 1,500 pounds per square foot and that a curved failure envelop passing through or nearly through the origin be fitted to the test results. The shear strength parameters used in the analysis should be represented by the tangent to the to the curved envelop at the effective normal stress being analyzed. Skempton and DeLory (1957) concluded there is "rather strong evidence suggesting that, on a geological time scale, stiff-fissured clays in natural slopes behave as if c'=0" even though their laboratory shear test data indicated an average cohesion of about 250 psf. Therefore, practitioners should be cautious when using cohesion in the infinite slope formula to determine factor of safety.

The infinite slope analysis method discussed above when combined with properly determined shear strength parameters represents an analysis that should accurately represent the worst case conditions for this type of failure. Therefore, the imposition of a required factor of safety of 1.5 is more conservative than for other types of analyses. It is the Committee's judgment that a lower factor of safety of 1.3 should be applied to analyses based on shear strength parameters determined from a failure envelope that passes through the origin. Should the governing agency not elect to use the recommended lower factor of safety, the analysis of the surficial stability of slopes that are 2:1 in gradient or flatter should not, in the Committee's judgment, be required unless local experience indicates that slopes at that gradient commonly experience surficial instability.

10 GROUND MOTION PARAMETERS FOR SEISMIC SLOPE STABILITY ANALYSES

The ground motion parameters used in the recommended seismic slope displacement analysis procedures (Chapter 11) are maximum horizontal acceleration (MHA), duration of strong shaking, and mean period of ground motion (T_m). Duration is typically quantified for this purpose as the time across which 90% of the energy in an earthquake accelerogram is released, or more specifically as the time between 5% and 95% normalized Arias Intensity ($D_{5\text{-}95}$). The ground motion parameters of MHA, $D_{5\text{-}95}$, and T_m are, in turn, functions principally of earthquake magnitude (M), site-source distance (r), site condition (i.e., rock vs. soil), and for MHA, style of faulting.

Consultants can perform either a site-specific seismic hazard analysis to estimate MHA, or they can use the moderately detailed CDMG seismic hazard maps. Seismic hazard maps for $D_{5\text{-}95}$ and T_m are not available, but those ground motion parameters can be estimated on a site-specific basis from probabilistic seismic hazard analyses or the deterministic procedures described below. It should be noted that a site-specific analysis of seismic hazard performed by an experienced earthquake engineer or seismologist would generally be expected to provide more accurate ground motion estimates than would the use of CDMG maps.

Guidelines for the estimation of MHA, $D_{5\text{-}95}$, and T_m are provided in the following sections. Guidelines for the selection of the design-basis magnitude (M) and distance (r) are also provided, as those parameters are needed for the estimation of $D_{5\text{-}95}$ and T_m.

10.1 GROUND MOTION ESTIMATION: GENERAL CONSIDERATIONS

There are two basic approaches for calculating site-specific design ground motion parameters: deterministic and probabilistic. In the deterministic approach, a specific scenario earthquake is selected (i.e., with a particular magnitude and location) and the ground motion is computed using applicable attenuation relations. For a given set of seismological parameters (i.e., magnitude and distance), attenuation relations provide a probabilistic distribution of ground motion described in terms of a median and standard deviation. Note that attenuation relations thus do not provide a specific value of the ground motion parameter. Therefore, even when a deterministic assessment of the causative earthquake is specified in terms of its magnitude and distance to the site, there is still a large range of potential ground motions that could occur as described by attenuation relations. Depending on the level of conservatism desired in deterministic analyses, typically either the median (50th percentile) or median-plus-one-standard-deviation (84th percentile) ground motion is used for design.

In the probabilistic approach, multiple potential earthquakes are considered. That is, all of the magnitudes and locations believed to be applicable to all of the presumed sources in an area are considered. Thus, the probabilistic approach does not consider just one scenario, but all of the presumed possible scenarios. Also considered are the rate of earthquake occurrence (how often each scenario earthquake occurs) and the probabilities of earthquake magnitudes, locations, and rupture dimensions. Moreover, the probabilistic approach considers all possible ground motions for each earthquake and their associated probabilities of occurring based on the ground motion attenuation relation.

The basic probabilistic approach yields a probabilistic description of how likely it is that different levels of ground motion will be exceeded at the site within a given time period, not merely how likely an earthquake is to occur. The inverse of the annual probability (i.e., the probability of exceedance for one year) is called the return period. Because probabilistic seismic hazard analyses sum the contribution of all possible earthquakes on all of the seismic sources presumed to impact a site, they do not result in a unique magnitude and distance that corresponds to the estimated acceleration value. Additional efforts are needed to extract the magnitude and distance most strongly contributing to the acceleration at a given hazard level. To estimate a magnitude and distance that can be paired with a given acceleration point (i.e., MHA and associated probability of exceedance), the hazard analysis for a given acceleration must be de-aggregated to develop the modal magnitude, $\overline{M}$, and modal distance, $\bar{r}$. Parameters $\overline{M}$ and $\bar{r}$ can be thought of as the magnitude and distance that contribute most strongly to the selected hazard level at the site. The process of de-aggregating the hazard to derive $\overline{M}$ and $\bar{r}$ is straightforward, but it must be understood that the de-aggregation is a function of hazard levels (i.e., different return periods). In addition, de-aggregation is sensitive to the ground motion parameter for which the hazard analyses are performed (i.e., different values of $\overline{M}$ and $\bar{r}$ could be obtained for MHA than for a long-period spectral acceleration).

There is a widespread misunderstanding of the relationship between deterministic and probabilistic analyses. Deterministic analyses are often (mistakenly) thought to provide "worst case" ground motions. That misunderstanding is a result of nebulous terminology that has been used in earthquake engineering. Terms such as "maximum credible earthquake" and "upper bound earthquake" are often used, which are intended to refer to the largest magnitude earthquake that the fault closest to the site is capable of producing (which may sound like a worst case). However, the definition of the largest magnitude earthquake for a given fault is often unclear, as magnitude is generally correlated to the length (or area) of a fault using regression equations, which have uncertainty. Accordingly, a real "worst case" magnitude needs to consider the standard deviation on the relationship between magnitude and fault size. Real "worst case" maximum magnitudes would need to be 2 to 3 standard deviations above the median magnitude corresponding to the assumed "known" fault size. Each standard deviation on fault size increases the estimated maximum magnitude by about 1/4 to 1/3 of a magnitude unit, depending on the regression equation being used. However, the number of standard deviations above the median is rarely provided in assessments of maximum credible earthquakes.

The evaluation of ground motions associated with deterministic earthquake scenarios (such as "maximum credible") introduces additional complexity due to the aforementioned fact that attenuation relations provide a distribution of ground motion, not a single value. When using attenuation relations for acceleration, each increase of standard deviation increases the estimated ground motion by a factor of 1.5 to 2 depending on the attenuation relation and the spectral period of the ground motion. Consequently, the resulting "worst case" ground motion is likely to be quite high. The cost of designing for such "worst case" ground motions would be very large, and more importantly, the chance of such ground motions occurring during the life of the structure is so small that, in most cases, to design for such rare events is unreasonable. As a result, most engineers consider it unnecessary to design for such "worst case" ground motions. But, the question of how much to back off from that "worst case" leads to the issue of acceptable risk (i.e., if you are not designing for the "worst case," what chance are you taking?). That, in turn, leads back to the need for probabilistic analysis to quantify the risk.

Although the Committee generally recommends the use of probabilistically defined ground motions, deterministically estimated ground motions may be used in engineering analyses provided the consultant and the cognizant regulatory agency agree to the manner in which the deterministic analysis should be performed. Limited deterministic "checks" on the results of the probabilistic analyses are also encouraged.

10.2 ESTIMATING MAXIMUM HORIZONTAL ACCELERATION (MHA)

Ground motion provisions in the Uniform Building Code (UBC), which forms the basis for most building design in California, are based loosely on a probabilistically derived spectral accelerations (including MHA) with a 10 percent probability of exceedance in 50 years (i.e., a 475-year return period). Accordingly, in order to perform a site-specific analysis that is consistent with the UBC, ground motions should be obtained from a probabilistic seismic hazard analysis (PSHA).

Probabilistic seismic hazard analyses can be performed on a site-specific basis using available commercial computer codes. Alternatively, available CDMG maps can be used to estimate accelerations at different hazard levels. The CDMG maps can be useful provided the hazard level of interest is represented on the maps, there are not unusual soil conditions that could significantly affect ground motions (such as soft clay or peat), and the seismic source modeling used by CDMG remains appropriate (i.e., additional fault information compiled since publication of the CDMG maps has not rendered them obsolete). Estimation of peak accelerations using the state maps or site-specific analyses are discussed below.

10.2.1 State Maps

Ground motion maps are being created for each area affected by the California Seismic Hazards Mapping Act as a by-product of the delineation of Seismic Hazards Zones by the Department of Conservation. They form the basis of earthquake shaking opportunity in the regional assessment of liquefaction and seismically-induced landslides for zonation purposes. The maps are generated at a scale of about 1:150,000, using the MapInfo® street grid as the base. The maps are produced using a data-point spacing of about 5 kilometers (0.05 degrees), which is the spacing that was used to prepare the small-scale state ground-motion map used for the Building Code (Petersen et al., 1996; Frankel, 1996; Petersen et al., 1999).

Ground motions shown on the maps are expressed as maximum horizontal accelerations (MHA) having a 10-percent probability of being exceeded in a 50-year period (corresponding to a 475-year return period) in keeping with the UBC-level of hazard. Separate maps are prepared of expected MHA for three types of surficial geology (hard rock, soft rock, and alluvium), based on averaged ground motions from three different attenuation relations. When using those maps, it should be kept in mind that each assumes that the specific soil condition is present throughout the entire map area. Use of a MHA value from a particular soil-condition map at a given location is justified by the soil class determined from the site-investigation borings.

The set also includes a map of modal magnitude and distance pairs (i.e., $\overline{M}$ and $\bar{r}$) calculated at the same grid spacing as MHA. Those values represent the de-aggregated 475-year hazard level, and are available for the ground motion parameter of MHA for an alluvial site condition (the parameters are not sensitive to site condition, and hence the values on the maps can also be used for rock and soft rock site conditions). Because of the discrete nature of de-aggregated hazard, the user is cautioned not to interpolate modal parameters to the project site location when using these maps. Instead, consideration should be given to the larger in the range of values at the four nearest grid nodes. Consideration should also be given to events larger than the modal values, which occur less frequently, but may have a longer duration and a greater influence on ground failure potential at the site. Because such events are not considered in the State maps, the Committee believes that site-specific PSHA with de-aggregation is the preferred method of developing input ground motions for the analysis of seismic landslide hazards.

The complete set of four ground motion maps prepared by the State of California is contained in the evaluation reports that correspond to each seismic hazard zone quadrangle map. Color images of seismic hazard zone maps, and the text of associated evaluation reports are accessible at the CDMG Web site found at the address:

http://www.consrv.ca.gov/dmg/shezp/map_data.htm.

10.2.2 Site-Specific Probabilistic Seismic Hazard Analyses

Results of probabilistic seismic hazard analysis can vary depending on the fault/attenuation models used as input to the analysis. Accordingly, whenever a site-specific probabilistic seismic hazard analysis is performed, the following information should be documented: seismic source parameters (including style of faulting, source dimensions, and fault slip rates), magnitude-recurrence relations (i.e., truncated exponential or characteristic earthquake model), and the ground motion attenuation relationships. Many of the seismic source parameters are documented on the CDMG web site and can readily be incorporated by consultants into their seismic hazard analyses. Any significant deviations from those parameters that are used in site-specific analyses should be explained and justified based on sound, new data.

As a final comment on probabilistic seismic hazard analyses, it should be noted that such analyses must incorporate the uncertainty in the attenuation relation (i.e., uncertainty as represented by the standard error term). Probabilistic seismic hazard analyses that neglect uncertainty in the attenuation are sometimes performed by consultants because in 1978 and 1983, the United States Geological Survey (USGS) set an incorrect example by not using the standard deviation on the attenuation function when they developed the United States national seismic hazard maps. However, in 1990 (MF-2120) and on subsequent work, the USGS corrected that practice and has properly incorporated uncertainty in attenuation relations in their seismic hazard analyses. Also, the State of California properly incorporates the uncertainty in the attenuation in their probabilistic seismic hazard analyses (Petersen et al., 1996).

10.2.3 Site-Specific Deterministic Analyses

Deterministic analyses can be used to evaluate the seismic demand that would be placed on a site if a specific earthquake were to occur. If deterministic seismic hazard analyses are to be used to develop ground motion estimates, the following should be clearly documented in the project report: definition of the scenario earthquake, attenuation relationship used to evaluate ground motions for the scenario earthquake, and the percentile ground motion (e.g., 50th, 84th, etc.) that was selected. The engineer may wish to consult with the reviewing agency in developing these criteria for deterministic analyses. For non-critical structures, many engineers have used median ground motions from attenuation relations based on characteristic magnitudes associated with nearby faults; whereas for critical structures, 84th percentile ground motions have sometimes been used. In a region where an individual fault dominates the seismic hazard, the level of uncertainty to be used in prescriptive deterministic analyses can be estimated by performing probabilistic analyses and comparing the results with deterministic analyses at different uncertainty levels.

10.3 OTHER GROUND MOTION PARAMETERS

As noted at the beginning of this chapter, three ground motion parameters are needed for the evaluation of seismic slope stability – MHA, duration of strong shaking ($D_{5\text{-}95}$), and mean period (T_m). Of those, only MHA maps are currently available from CDMG. The focus of this section, therefore, is the estimation of $D_{5\text{-}95}$ and T_m for seismic slope displacement calculations.

The parameters $D_{5\text{-}95}$ and T_m are functions of magnitude (M), distance (r), and site condition (S=0 for rock, S=1 for soil). For a given M, r, and S, regression equations are available that provide a log-normal distribution of the $D_{5\text{-}95}$ and T_m parameters, not a single value. For use with the seismic slope displacement methodology discussed in Section 11.2, median values of $D_{5\text{-}95}$ and T_m can be used. Those values should be evaluated for the $\overline{M}$, $\bar{r}$ magnitude-distance pair (where $\overline{M}$ and $\bar{r}$ represent the 475-year hazard level for MHA). At their discretion, consultants may also wish to consider additional scenario earthquakes with larger magnitudes that might occur on major faults near the site. Once a magnitude-distance pair has been selected, median values of $D_{5\text{-}95}$ and T_m can be calculated as follows:

Duration (Abrahamson and Silva, 1996)

Median values of $D_{5\text{-}95}$ on rock can be estimated as follows. For r >10 km,

$$\ln(D_{5-95})_{med} = \ln\left[\frac{\left(\frac{\exp[5.204+0.851\cdot(M-6)]}{10^{1.5M+16.05}}\right)^{-1/3}}{15.7*10^{6}} + 0.063\cdot(r-10)\right] + 0.8664 \qquad (10.1a)$$

For $r < 10$ km

$$\ln(D_{5-95})_{med} = \ln\left[\frac{\left(\frac{\exp[5.204+0.851\cdot(M-6)]}{10^{1.5M+16.05}}\right)^{-1/3}}{15.7*10^{6}}\right] + 0.8664 \qquad (10.1b)$$

Mean Period (Rathje et al., 1998)

Rathje et al. (1998) define mean period (T_m) as the inverse of the weighted average frequency, with weights defined from the Fourier amplitudes spectrum over a frequency range of 0.25 to 20 Hz. For practical application, that parameter can be estimated as,

$$\ln(T_m) = \ln(C_1 + C_2\cdot(M-6) + C_3\cdot r) \qquad M \le 7.25 \qquad (10.2a)$$

$$\ln(T_m) = \ln(C_1 + 1.25\cdot C_2 + C_3\cdot r) \qquad 7.25 \le M \le 8.0 \qquad (10.2b)$$

where parameters C_1, C_2, and C_3 should be selected for a rock site condition using the parameters in Table 10.1.

Table 10.1: Coefficients for Estimating T_m.

C_1	C_2	C_3	ε_T
0.411	0.0837	0.00208	0.437

11 SEISMIC SLOPE STABILITY ANALYSIS

11.1 INTRODUCTION

11.1.1 Background

Recent practice for analysis of seismic slope performance has been to use a pseudo-static representation of seismic loading in a conventional limit-equilibrium analysis, or to perform a displacement analysis based on the analogy of a rigid block on an inclined plane (i.e., Newmark-type displacement analysis; Newmark, 1965).

There are two elements associated with a pseudo-static slope stability analysis procedure. First, a horizontal destabilizing seismic coefficient (k) must be specified, which represents the fraction of the weight of the slide mass that acts horizontally through the centroid of the mass. Second, a minimum acceptable factor of safety must be specified for the slope with the pseudo-static seismic force applied to it. In southern California, the most commonly used pseudo-static procedure is one adopted by Los Angeles County, and is modified from the recommendations of Seed (1979). The Seed procedure calls for k = 0.15 and FS ≥ 1.15, and was calibrated from Makdisi and Seed (1978) displacement analyses so as to produce slope deformations of one meter during magnitude 8.25 earthquakes. LA County has modified this procedure to have k = 0.15 and FS ≥ 1.10. Pseudo-static methods are recommended herein for the purpose of a screen analysis for slopes within hazard zones. However, the recommended procedures for screen analyses are modified from the Seed criterion to more properly account for the effects of seismicity on slope deformation hazard, and to recognize the relatively small deformation tolerance of typical hillside construction. These procedures are described in Section 11.2.

Newmark-type displacement analyses can be performed with two general methods. The first involves formal numerical integration of time histories of shaking within a slide mass according to the procedure described by Franklin and Chang (1977). The second method makes use of correlations between calculated Newmark displacements, selected ground motion parameters, and the ratio of seismic load resistance to peak demand (k_y/k_{max}, see definitions below). Several such correlations are available, including Makdisi and Seed (1978) and Bray and Rathje (1998).

The use of a displacement analysis is recommended for slopes that fail the screen. Various approaches for calculating Newmark displacements are discussed in Section 11.3.

11.1.2 Overview of Recommended Analysis Procedure

An analysis of seismic slope stability must include the following steps:

1. Characterize the site geometry and stratigraphy, using appropriate field testing techniques such as borings with sampling and/or CPT soundings.

2. Evaluate soil material strengths for dynamic conditions, as described in Chapter 7 of this report.

3. Analyze the maximum horizontal acceleration (MHA) at the site for a rock site condition and a 475-year hazard level. Analyze the mode magnitude ($\overline{M}$) and mode site-source distance ($\bar{r}$) of the earthquake sources most significantly contributing to the 475-year MHA hazard level. Ground motion hazard analyses of this type are described in Section 10.2.1.

4. Analyze possible liquefaction hazards (see Liquefaction Hazards Committee report, Martin and Lew, 1999). If soil liquefaction will be triggered, post-liquefaction residual strengths should be used in lieu of the material characterization in (2).

5. Perform a screen analysis for seismic slope performance (Section 11.2).

6. For sites failing the screen analysis, evaluate median values of the significant duration and mean period associated with $\overline{M}$ and $\bar{r}$, as described in Section 10.3.

7. For sites failing the screen analysis, perform slope displacement analysis (Section 11.3).

This chapter will focus on Steps 5 through 7 listed above.

The following nomenclature is used:

MHA = Maximum Horizontal Acceleration expected at the site.

$D_{5\text{-}95}$ = Significant duration of shaking, i.e., 5-95% normalized Arias intensity (sec).

$\overline{M}$, $\bar{r}$ = Mode magnitude and distance (in km) of causative earthquakes (based on de-aggregation of MHA hazard corresponding to a 475-year return period).

k_y = Yield acceleration of slope.

MHEA = Maximum Horizontal Equivalent Acceleration (see Section 11.3)

k_{max} = MHEA/g.

T_m = mean period of input rock motion (sec)

T_s = fundamental period of equivalent 1-D slide mass at small strains (sec)

u = calculated slope displacement (in cm)

11.2 SCREENING ANALYSIS

11.2.1 Background

Seismic Hazard Zone maps published by the CDMG include Landslide Hazard Zones. Analyses of the type described in this chapter are required for sites located within those zones. The purpose of these analyses is to determine if the site has a significant seismic slope deformation potential. The mere fact that a site is within a Landslide Hazard Zone does not mean that there necessarily is a significant landslide potential at the site, only that a study should be performed to determine the potential.

The SP 117 Guidelines state that an investigation of the potential seismic hazards at a site can be performed in two steps: (1) a screening investigation and (2) a quantitative evaluation. The purpose of the screening investigation for sites within zones of required study is to filter out sites that have no potential or low potential for landslide development.

The screening criteria described in Sections 11.2.2 to 11.2.3 below may be applied to determine if further quantitative evaluation of landslide hazard potential is required. If the screening investigation clearly demonstrates the absence of seismically induced landslide hazards at a project site and the lead agency technical reviewer concurs, the screening investigation will satisfy the site investigation report requirement for seismic landslide hazards. If not, a more thorough quantitative evaluation will be required to assess the seismic landslide hazard, as described in Section 11.3.

11.2.2 Development of Screening Analysis Procedure

The screening analysis procedure recommended herein is based on a pseudo-static representation of seismic slope stability. The procedure is implemented by entering a horizontal seismic coefficient (*k*) into a conventional slope stability calculation. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the mass. If the factor of safety is greater than one (FS > 1), the site passes the screen, and the site fails if FS < 1.

The seismic coefficient to be used in the analyses is taken as,

$$k_{eq} = f_{eq} \times \left(MHA_r / g \right) \qquad (11.1)$$

where MHA_r is the maximum horizontal acceleration at the site for a soft rock site condition; g = acceleration of gravity; and f_{eq} is a factor related to the seismicity of the site, as described below.

For a given MHA_r, large magnitude earthquakes will tend to cause poorer slope performance than smaller magnitude earthquakes. One important reason for this is that large magnitude earthquakes have longer durations of shaking. Previous pseudo-static procedures for seismic slope stability have specified a single value for f_{eq}, and thus have made implicit, and usually very conservative, assumptions about the magnitude of earthquakes causing the design-basis MHA_r. This committee has sought to reduce unnecessary conservatism by developing a range of f_{eq} values that are a function of magnitude (as represented by $\overline{M}$) and site-source distance (as represented by $\bar{r}$). The development of those f_{eq} values is described in detail in Appendix A, and is synthesized briefly in the following (see also Stewart et al., 2002).

Magnitude- and distance-dependent f_{eq} values were developed using a model for seismic slope displacements based on a Newmark-type analysis. Bray and Rathje (1998) have found that Newmark displacements (u) are a function of k_y/k_{max}, k_{max}, and $D_{5\text{-}95}$. Bray and Rathje present a relationship to predict the median value of slope displacement that would be expected given the above parameters, as well as the standard error (the relationship is log-normally distributed).

The Bray and Rathje model was used to relate magnitude, distance, and MHA to f_{eq} based on the following assumptions and observations:

1. Factor f_{eq} was related to k_y/k_{max}. The rationale for this relationship is described in detail in Appendix A.

2. Two values of threshold Newmark displacement were used - 5 cm and 15 cm. The use of two threshold displacements is intended to enable engineers and regulatory agencies to exercise judgment on the level of performance they wish to enforce. However, the recommendation of the majority of the Committee is that the screen procedure based on 5 cm threshold displacement be used for typical hillside construction. A minority of the Committee felt that the screen procedure based on 15 cm threshold displacement is more appropriate. In either case, it should be noted that Newmark displacements provide only an index of slope performance. The 5 cm value likely distinguishes conditions in which very little displacement is likely from conditions in which moderate or higher displacements are likely. The 15 cm value likely distinguishes conditions in which small to moderate displacement are likely from conditions where large displacements are likely. Further discussion on threshold Newmark displacements is provided in Section 11.3.4.

3. Factor k_{max} is related to $MHA_r \times NRF/g$, where NRF is a factor that accounts for the nonlinear response of the materials above the slide plane. Parameter $D_{5\text{-}95}$ is a function of magnitude and distance, as discussed in Section 10.3.

Based on the above, calculations were performed to evaluate for various combinations of MHA_r, magnitude, and distance, the f_{eq} values that cause the probability that seismic slope displacement would exceed 5 cm or 15 cm to be 50%. The Committee chose to use a 50% probability level because we believed probabilities departing significantly from 50% could significantly bias the effective return period from the standard 475-year hazard level. Additional details on this calculation are provided in Appendix A. The results of the calculations are shown in Figures 11.1(a) and 11.1(b) for the 5 cm and 15 cm threshold displacements, respectively.

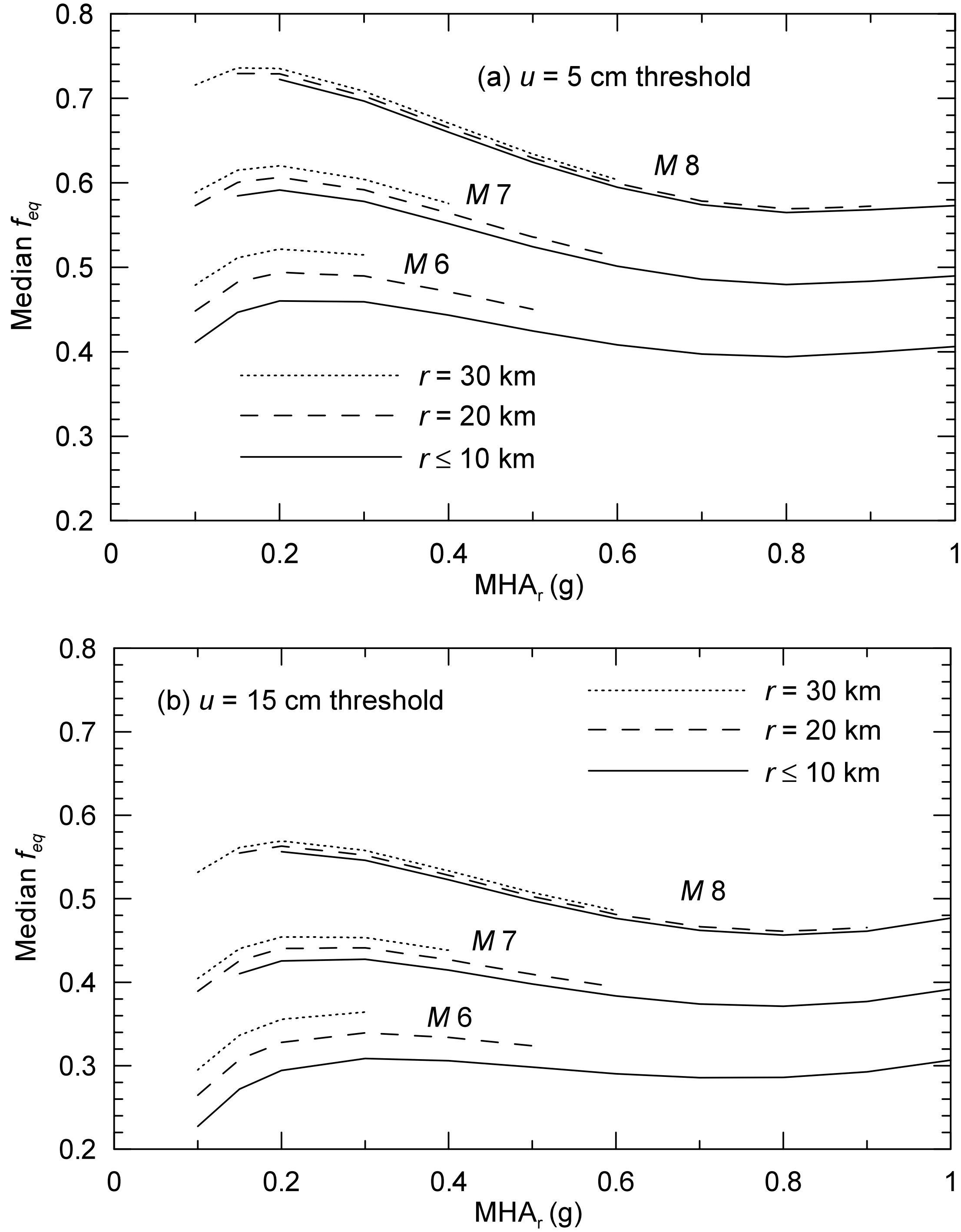


Figure 11.1. Required Values of f_{eq} as Function of MHA$_r$ and Seismological Condition for Threshold Displacements of (a) 5 cm and (b) 15 cm

The equation of the curves in Figure 11.1 is as follows:

$$f_{eq} = \frac{NRF}{3.477} \times \left[1.87 - \log_{10}\left(\frac{u}{(MHA_r / g) \times NRF \times D_{5-95}} \right) \right] \tag{11.2}$$

where u is in units of cm, $D_{5\text{-}95}$ = median duration (in seconds) from Abrahamson and Silva (1996) relationship (defined in Eq. 10.1) and NRF is defined by the relationship tabulated subsequently in Figure 11.2, which can be approximated by:

$$NRF \approx 0.6225 + 0.9196 \times Exp\left(\frac{-MHA_r / g}{0.4449} \right) \tag{11.3}$$

for $0.1 < MHA_r/g < 0.8$.

11.2.3 Screening Criteria

In summary, the following procedure is recommended for performing screening analyses for seismic slope stability:

1. Set up an analytical model for the slope as would normally be done for a static application, but with soil strengths that are appropriate for dynamic loading conditions. As noted in Chapter 7, this may require that different drainage conditions be considered than in the static case, and also requires consideration of rate effects and cyclic degradation on soil strength.

2. Use the procedures in Section 10.2 to estimate the maximum horizontal acceleration at the location of the site for a rock site condition (MHA_r). Parameter MHA_r should generally be evaluated using probabilistic seismic hazard analysis for a 475-year return period. Identify the mode magnitude ($\overline{M}$) and mode distance ($\bar{r}$) from de-aggregation of that hazard level.

3. Evaluate the site seismic coefficient using the procedures described in Section 11.2.2 with a value of threshold displacement that is considered acceptable by the local regulatory agency.

4. Perform a pseudo-static calculation of slope stability using the seismic coefficient from (3), and find the minimum factor of safety. Note that the critical failure surface will generally be shallower than the critical surface without a seismic coefficient.

5. Denote the factor of safety from (4) as FS. If FS > 1, the site passes the screen. However, for critical projects, consultants may want to perform additional checks for specific, large seismic sources in the local area, calculating M and r for each source deterministically. For each source considered, one would evaluate MHA_r and f_{eq} deterministically, and then check

the FS. The need for such deterministic checks must be made on a project-specific basis by the design engineer and cognizant public official. If FS < 1, the site fails the screen, and the analyses in Section 11.3 should be used.

11.3 SLOPE DEFORMATION ANALYSIS

Slope deformation analyses require the estimation of yield acceleration (k_y) and a horizontal equivalent acceleration (HEA or MHEA), which represents the severity of shaking within the slide mass. The focus of Sections 11.3.1 and 11.3.2 below is on the evaluation of these two factors. The evaluation of expected displacement given these factors is then discussed in Section 11.3.3.

11.3.1 Evaluation of Yield Acceleration (k_y)

A pseudostatic analysis is performed using static limit equilibrium slope stability procedures to determine the yield acceleration (k_y). Various assumed values of horizontal (pseudostatic) acceleration (representing the earthquake shaking) are applied, and the smallest value that reduces the factor of safety against sliding to unity is taken as k_y. The critical surface identified with this procedure will generally be slightly shallower than the critical surface identified without a seismic coefficient. The most critical surface for seismic slope displacement analysis may be shallower still, as described further in Section 11.3.3, and hence k_y values for surfaces shallower than that providing the minimum k_y should also be evaluated. Guidelines on the selection of these shallower surfaces are presented in Section 11.3.3.

In the evaluation of k_y, it is critical that soil strengths used in the analyses be appropriate for dynamic loading conditions. As noted in Chapter 7, this may require that different drainage conditions be considered than in the static case, and also requires consideration of rate effects and cyclic degradation effects on soil strength. It should also be noted that stability calculations for k_y should utilize total unit weights with boundary water pressures (if present), and not buoyant weights. This is necessary because earthquake inertial loads apply to the total weight of the sliding mass.

11.3.2 Evaluation of Seismic Demand in Slide Mass

A seismic demand evaluation for slope displacement analysis generally begins with a probabilistic evaluation of MHA_r, $\overline{M}$ and $\bar{r}$ corresponding to a 475-year return period (Section 10.2). In some circumstances, consultants may wish to supplement these probabilistically-derived parameters with deterministic evaluations of MHA_r for a scenario event with a specific magnitude occurring at a specific distance from the site. The procedures that follow describe how these

MHA-*M-r* parameters can be translated into a more useful representation of demand for slope stability analysis.

The seismic loading for a potential sliding mass can be represented by the horizontal equivalent acceleration, HEA. HEA/*g* represents the ratio of the time-dependant horizontal inertia force applied to a slide mass during an earthquake to the weight of the mass. For a horizontal slide plane and horizontal ground surface, HEA can be calculated as:

$$HEA(t) = \left(\frac{\tau_h(t)}{\sigma_v} \right) g \tag{11.4}$$

where *t* indicates that there is time variation, τ_h is the horizontal shear stress at the depth of the sliding surface calculated by a one-dimensional seismic site response analysis program (e.g., SHAKE91, Idriss and Sun, 1992; D-MOD, Matasovic, 1993), and σ_v is the total vertical stress at the depth of the sliding surface. For more complex geometries (i.e., not one-dimensional), a rigorous analysis of HEA requires the use of two-dimensional finite element analyses (e.g., QUAD4M; Hudson et al., 1994). Rathje and Bray (1999a) have found that 1-D analyses generally provide a conservative estimate of HEA(*t*) for deep sliding surfaces and a slightly unconservative estimate for shallow surfaces near slope crests. MHEA is the maximum horizontal equivalent acceleration over the duration of earthquake shaking. For slope displacement analyses, seismic demand is typically represented by HEA time histories or MHEA coupled with duration $D_{5\text{-}95}$.

The seismic demand in a slide mass can be relatively rigorously evaluated from two dimensional finite element dynamic response analyses using a program such as QUAD4M (Hudson et al., 1994). Those analyses enable the evaluation of HEA time histories that are customized to the specific geometry and soil condition at the subject site. The analyses should be performed using sets of at least 5-10 time histories as input. Those time history sets should be appropriate for the magnitude and site-source distance that control the site hazard. Fewer time histories (3-4) can be used if they are scaled to match the constant hazard spectrum for the site (established from a site-specific probabilistic seismic hazard analysis) across the period range of interest (e.g., Richardson et al., 1995; Kavazanjian et al., 1997). Further discussion on time histories for slope displacement analyses is provided in Section 11.3.3.

A second procedure represents the amplitude of seismic demand with MHEA. The procedure was developed by Bray et al. (1998) from statistical analysis of many wave propagation results in equivalent one-dimensional slide masses. The procedure normalizes MHEA in the slide mass by the product of MHA_r and a nonlinear response factor (NRF). Parameter NRF accounts for nonlinear ground response effects as vertically propagating shear waves propagate upwards through the slide mass. The normalized acceleration is then related as shown in Figure 11.2 to the period of the sliding mass (T_s) normalized by the mean period of the input motion (T_m). It should be emphasized that MHA_r to be used with Figure 11.2 represents the MHA that would be expected at the site using a soft rock site condition (regardless of the actual soil condition present beneath the slide mass). The quantity T_m represents the mean period of the earthquake and can be estimated from magnitude and distance using relations provided in Section 10.3. Finally, T_s represents the fundamental period of the sliding mass, which can be taken as:

$$T_s = \frac{4H}{V_s} \tag{11.5}$$

where H = maximum vertical distance between the ground surface and slip surface used to determine k_y (e.g., Figure 11.3), and V_s = representative small-strain shear wave velocity of materials above sliding mass. V_s can be measured in situ or can be estimated using published correlations (e.g., Tinsley and Fumal, 1985; Seed et al., 1984; Wills and Silva, 1998). When V_s varies as a function of depth within the materials above the slide plane, an average value can be estimated as:

$$V_s = \frac{\sum_i (V_s)_i \cdot h_i}{H} \tag{11.6}$$

where $(V_s)_i$ and h_i represent the shear wave velocity and thickness of layers within the slide mass, respectively.

For automated applications, the following equation represents the mean curve in Figure 11.2:

$$\ln\left(\frac{MHEA}{MHA_r \cdot NRF}\right) = -0.624 - 0.7831 \cdot \ln\left(\frac{T_s}{T_m}\right), \text{ for } T_s/T_m >\sim 0.5 \tag{11.7}$$

The standard deviation of the data in Figure 11.2 is 0.298. The ratio $MHEA/(MHA_r \times NRF)$ from Figure 11.2 need not be taken as larger than unity.

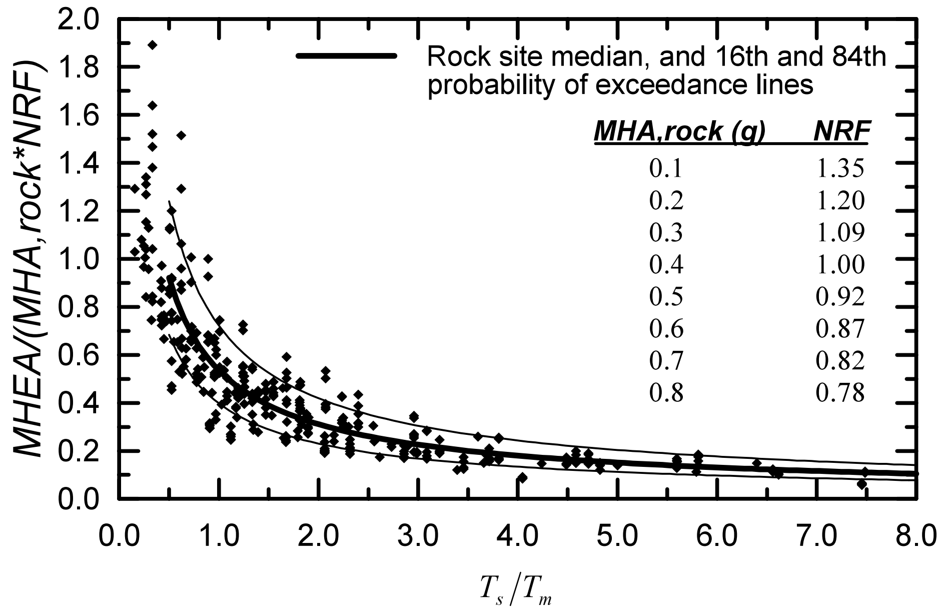


Figure 11.2. Normalized MHEA for Deep-Seated Slide Surface Vs. Normalized Fundamental Period of Slide Mass (after Bray et al., 1998).

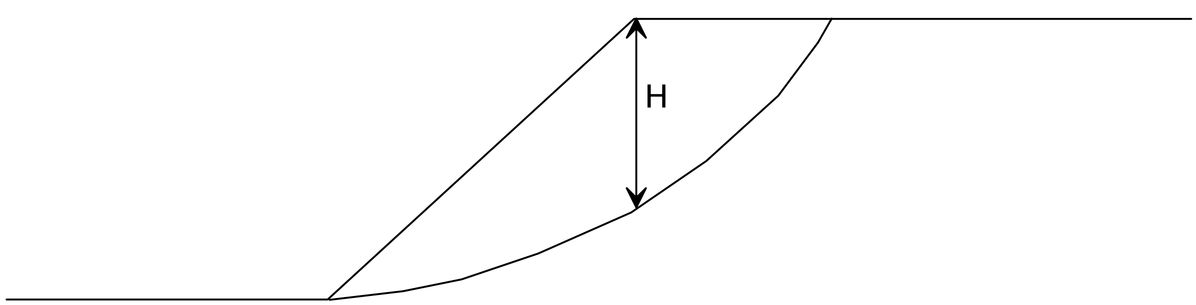


Figure 11.3. Definition of Height of Slide Mass for Use in Equation 11.5

11.3.3 Estimation of Seismic Slope Displacements

Two possible quantifications of demand for slope stability calculations were described in Section 11.3.2:

- Use of a simplifying assumption to evaluate MHEA = $k_{max} \cdot g$.
- Use of dynamic analysis to define time histories of horizontal equivalent acceleration, HEA(t).

In this section, three methods are described to evaluate seismic slope displacements. The first method utilizes MHEA to characterize the amplitude of shaking within the slide mass and D_{5-95} to characterize the duration. Normalized displacements, defined as $u/(k_{\max} \cdot D_{5-95})$ are related to k_y/k_{max} as shown in Figure 11.4. D_{5-95} is calculated using Eq. 10.1 (using $\overline{M}$ and $\bar{r}$).

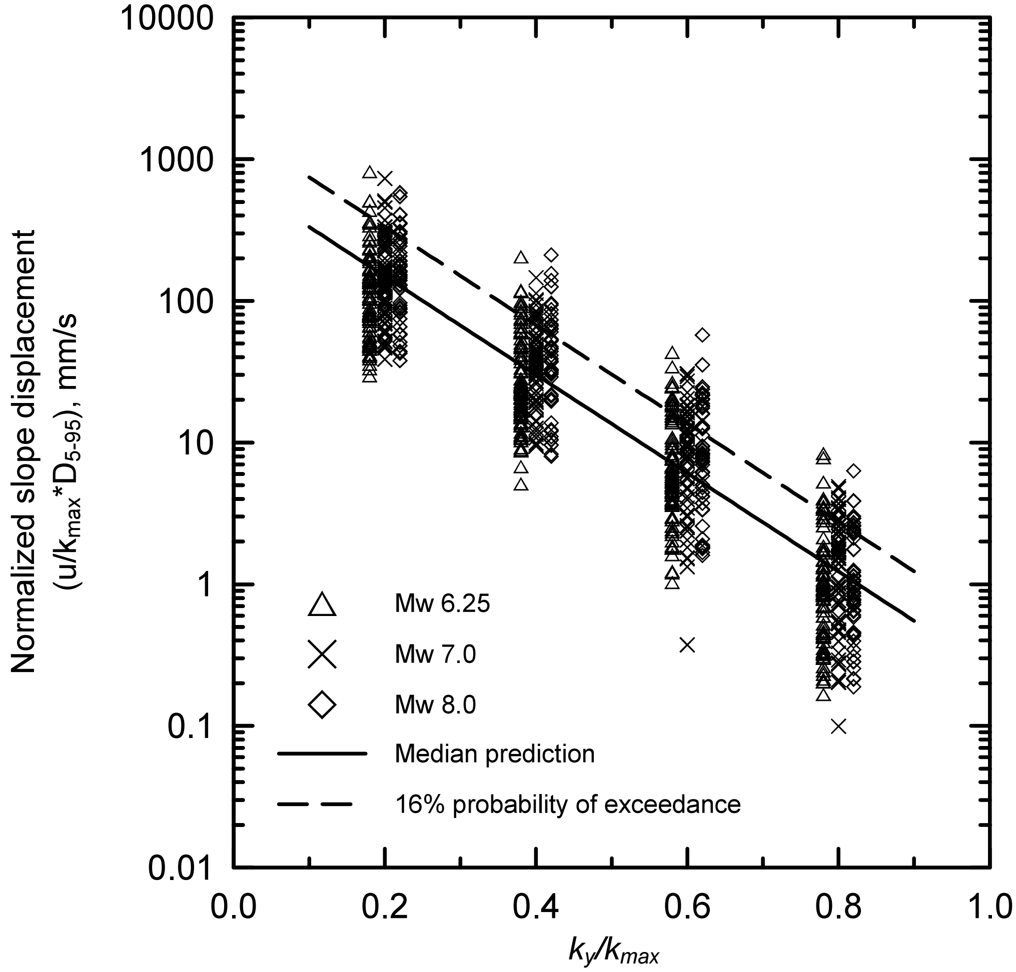


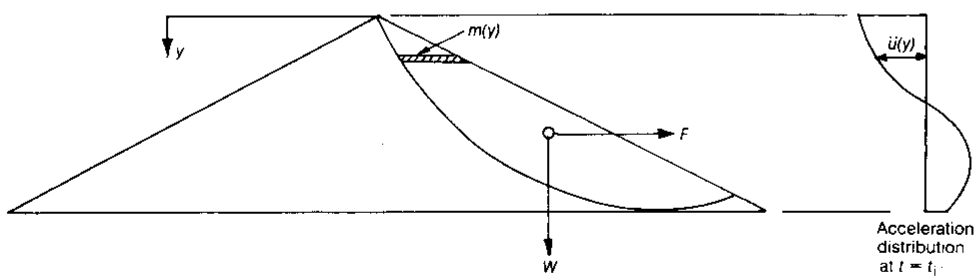
Figure 11.4. Normalized Sliding Displacement (modified from Bray et al., 1998).

It should be noted that a single, deterministic value of displacement is not obtained by this procedure, but rather a log-normal distribution of displacement. The Committee recommends the use of the median of this log-normal distribution, which is indicated by the solid line in Figure 11.4, and can be represented by the following equation:

$$\log_{10}\left(\frac{u}{k_{\max} \cdot D_{5-95}}\right) = 1.87 - 3.477 \cdot \frac{k_y}{k_{\max}} \tag{11.8}$$

where u is the median displacement in cm. The standard error is 0.35 in $\log_{10}$ units.

The second method for estimating slope displacement utilizes the recommendations of Makdisi and Seed (1978) for relating k_y/k_{max} to displacement u. Parameter k_{max} for application in the Makdisi and Seed procedure is not evaluated using the methods described in Section 11.2.2. Rather, the MHA at the crest of a triangular embankment section is evaluated, and k_{max} is estimated using Figure 11.5. The Committee is not aware of simplified procedures for evaluating the crest MHA for typical fill slope geometries, which are not triangular in cross-section. Such an evaluation would need to consider ground response effects through the slide mass and topographic effects. A consultant using the Makdisi and Seed approach should reach an agreement with the cognizant public official regarding an appropriate procedure for evaluating this crest acceleration, as well as a procedure for evaluating k_{max} from crest acceleration for non-triangular slope geometries.



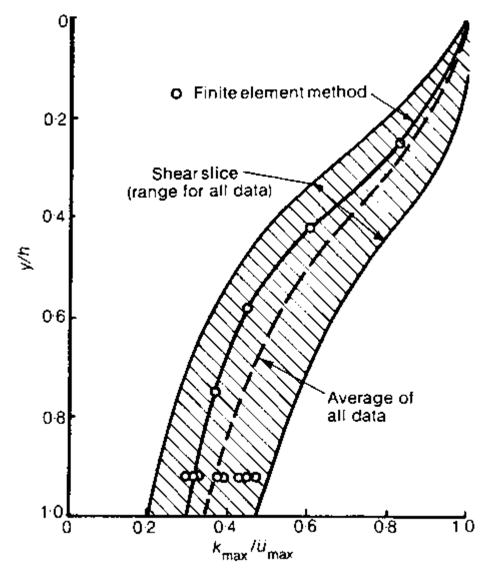


Fig. 11.5. Variation of k_{max} with Depth in Triangular-Shaped Embankment Section (Makdisi and Seed, 1978). Parameter $\ddot{u}_{max}$ Denotes Peak Acceleration at Embankment Crest.

With k_{max} evaluated in this way, and k_y evaluated per the recommendations of Section 11.3.1, slope displacement is estimated by the Makdisi and Seed procedure using Figure 11.6. No statistical quantities or equations are available to describe the range in Figure 11.6. The engineer and cognizant public official will need to reach agreement on what segment of the range is appropriate for a given problem. The range shown for magnitude 7.5 should be used when the design magnitude is greater than 6.7.

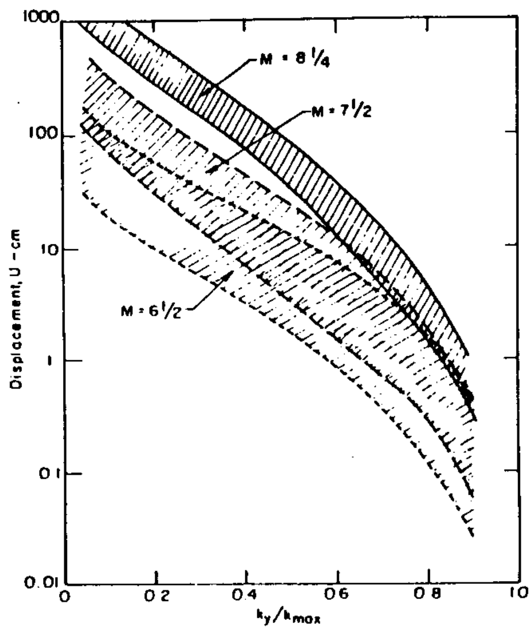


Fig. 11.6. Variation of Slope Displacement with Magnitude and k_y/k_{max} (Makdisi and Seed, 1978).

The third method for estimating slope displacement consists of performing Newmark-type integration analyses using HEA time histories and a k_y value. The procedures by which these analyses are performed are discussed in Newmark (1965) and Franklin and Chang (1977). Commercial computer codes for performing such analyses are available. As noted previously in Section 11.3.2, these analyses should be performed using at least 5-10 time histories of HEA, thus providing an equivalent number of displacement estimates from which a distribution can be formed (fewer time histories can be used if they are spectrally matched to a constant hazard spectrum from a site-specific probabilistic seismic hazard analysis). The engineer and cognizant public official will need to reach agreement on what percentile value of displacement is appropriate, given the project importance and the level of conservatism employed during other stages of the analysis.

As noted previously in Section 11.3.2, Newmark displacement analyses should generally be performed using HEA time histories, because such motions account for the effects of ground motion amplification and incoherence through the slide mass. However, there are a limited number of cases where Newmark analyses can be performed using as-recorded accelerograms as estimates of HEA time histories. As recommended by Rathje and Bray (1999b), this practice is acceptable for very short period slide masses having $T_s/T_m < 0.2$.

Finally, it should be noted that the identification of the most critical slip surface for seismic slope displacement analysis depends not only on the slope/material properties (as is the case under static conditions), but also on the variation of shaking in the slope. What is desired is the k_y/k_{max} combination that yields the largest slope displacement. In many cases, this will be the critical surface identified from the calculations described in Section 11.3.1. Shallower surfaces should be checked, however, because while they will have higher k_y values, they may also have larger k_{max} values, which could lead to larger displacements. The Committee considers the use of shallower surfaces to be unnecessary if $\mathrm{MHEA}/(\mathrm{MHA}_r \times \mathrm{NRF}) = 1.0$. However, if $\mathrm{MHEA}/(\mathrm{MHA}_r \times \mathrm{NRF})$ is less than 1.0 (see Figure 11.2), at a minimum, one additional surface should be considered and it is the deepest surface that produces $\mathrm{MHEA}/(\mathrm{MHA}_r \times \mathrm{NRF}) = 1.0$ (note that this will be shallower than the surface having the lowest k_y).

11.3.4 Tolerable Newmark Displacements

The final step in the analysis is to decide if the calculated displacement is acceptable. Ideally, allowable displacements for analyses would be established from a database in which observed slope displacements from earthquakes are correlated to measures of damage in structures associated with the slope displacements. Unfortunately, however, such data do not exist in sufficient quantity to be useful, and hence there is no rational basis for selecting allowable displacements. Accordingly, allowable displacement levels are established from engineering judgment. The judgment of the majority of the Committee is that if the critical slip surface from slope stability analyses daylights within a structure that is likely to be occupied by people during an earthquake, the median displacements (u) should be maintained at less than 5 cm. A minority of the Committee feels that those displacements through occupied structures should be maintained at less than 15 cm. Neither of these values (5 or 15 cm) is necessarily the "correct" value, because they are judgment-based. Individual agencies may wish to select their own allowable displacement values based on their experience and judgment. No matter which allowable displacement values are selected, the procedures described in the preceding sections can be readily applied with those threshold displacements.

The scope of this Committee's activities, and the Seismic Hazards Mapping Act, does not extend beyond inhabited structures. However, owners, engineers, or cognizant public officials may, at their discretion, wish to design for seismic slope stability in other portions of project sites as well. A majority of the Committee offer the following suggestions for such cases:

- For slip surfaces intersecting stiff improvements (such as buildings, pools, etc.), computed median displacements should be maintained at < 5 cm.

- For slip surfaces occurring in ductile (i.e., non strain softening) soil that do not intersect engineered improvements (e.g., landscaped areas and patios), computed median displacements should be maintained at < 15 cm.

- For slip surfaces occurring in soil with significant strain softening (i.e., sensitivity > 2), if k_y was calculated from peak strengths, displacements as large as 15 cm could trigger strength reductions, which in turn could result in significant slope de-stabilization. For such cases, the design should either be performed using residual strengths (and maintaining displacements < 15 cm), or using peak strengths with displacements < 5 cm. Further discussion of materials that may be subject to strain softening is presented in Section 7.2.2.

The slope displacements analysis methods described herein are simplified models that simulate slope deformations using the sliding of a block on an inclined plane. This model may be reasonable for slopes with narrow, well-defined slip surfaces. However, for slopes in which deformations are distributed across relatively broad zones, the analyses provide only an index of performance. It also should be noted that the displacements calculated here are best interpreted as occurring tangent to the slip surface, and thus will generally involve both horizontal and vertical components of movement. Finally, it is very important to note that analyses of the type discussed here only simulate deformations arising from permanent shear deformation in soil. Another significant source of deformation in earth fills is seismic compression (Stewart et al., 2001), which as noted in Section 1.3, has not been addressed by this document.

12 SLOPE STABILITY HAZARD MITIGATION

Slopes that possess factors of safety less than required by the governing agency, or with unacceptably large seismic slope displacements, require avoidance or mitigation to improve their stability. Even if a slope is found from analyses to be stable, it might require protection in order to avoid degradation of shear strengths from weathering, to remain stable under future increased loading conditions, to prevent toe erosion, or to remain stable under future, potentially higher groundwater conditions than assumed in the analyses. Protection for adjacent pad areas may also be required to minimize hazard from erosion and falling debris.

The most common methods of mitigation are (1) hazard avoidance, (2) grading to improve slope stability, (3) reinforcement of the slope or improvement of the soil within the slope, and (4) reinforcement of the structure built on the slope to tolerate the anticipated displacement. Avoidance involves placing a proposed improvement a sufficient distance from an unstable slope. Grading methods commonly employed to improve slope stability include partial or complete replacement of unstable soil. Slopes can be strengthened with soil reinforcement, retaining walls, deep foundations, geosynthetics, and/or soil nails/tiebacks can be used alone or in conjunction with grading to improve slope stability. Soil can be improved with cement or lime stabilization. Structures built on slopes also can be sufficiently reinforced to reduce damage to a tolerable amount. In addition, structures can be effectively isolated from ground deformations through the use of piles or compaction grouting.

The mitigation measures chosen for a given slope must be analyzed recognizing that different mitigation measures require analyses for different modes of failure. Some methods (for example, slope reinforcement) require consideration of strain compatibility and soil/structure and/or soil material interaction issues. The following sections describe both stabilization and mitigation measures, and the potential modes of failure that should be analyzed.

12.1 AVOIDANCE

The simplest method of mitigation may be to avoid construction on or adjacent to a potentially unstable slope. A setback distance for structures or other improvements/uses can be established from the slope such that failure of that slope would not pose a danger to site improvements. The setback distance is based on the slope configuration, probable mode of slope failure, factor of safety, and potential consequences of failure. Where feasible, an estimate of the "runout" that would occur in the event of a slope failure should be made. The required setback cannot generally be accurately calculated, therefore a large degree of engineering/geologic judgment is required.

12.2 GRADING

Grading can often be performed to entirely or partially remove potentially unstable soil to create a finished slope with the required factor of safety. The available grading methods range from reconfiguration of the slope surface to a stable gradient, to removal and recompaction of a soil that is preferentially weak in an unfavorable direction and its replacement with a more homogeneous soil with a higher strength.

12.2.1 Reconfiguration

The stability of a slope can be improved by reducing the driving forces as a result of flattening the slope and/or decreasing its height. The reconfigured slope must be analyzed and must have at least the minimum required factor of safety or less than the maximum allowable seismic displacement (see Section 9.3 for potential failure modes to consider).

12.2.2 Removal and Replacement

It may, in some cases, be feasible to completely excavate (remove) earth materials that contribute to the instability of a slope and replace the excavated soil with higher-strength materials that result in a slope with the minimum required factor of safety. Materials that typically contribute to slope instability, and can often be completely removed, include slopewash (colluvium) and landslide debris. Complete removal of an active landslide does not preclude the possibility of deeper seated sliding, which also should be checked in the analysis. The slope created should be analyzed for internal stability (within the replaced soil mass) and external stability (through the remaining native soil) Often, the excavated material is reused as fill, although, in some instances, new soil must be imported, if the strength of the existing soil when recompacted is inadequate. The compacted fill should be keyed and benched into competent material.

Creation of a temporary backcut is usually required when performing partial or total removal and replacement. The backcut must be analyzed and designed to have a sufficient static factor of safety during construction, typically 1.25, to allow the safe construction of the permanent slope

12.2.3 Stability Fills

A stability fill is used where a slope has an adequate factor of safety for gross stability, but an insufficient factor of safety for surficial stability or where the materials exposed at the slope surface are prone to erosion, sloughing, rock falls, or other surficial conditions that require remediation. Stability fills are relatively narrow, typically about 10 to 15 feet wide. Soil placed in the stability fill should be compacted to at least 90 percent of the maximum density as determined by ASTM D1557, unless a different degree of compaction is recommended by a Geotechnical Engineer and approved by the governing agency. Water content also should be controlled during compaction, because fills compacted to water contents wetter than the line of optimums have been shown to perform significantly better than fills compacted to lower water contents in both static and seismic conditions (Lawton et al., 1989; Whang, 2001). A higher percent relative compaction may be required for steeper slopes and coarse-grained soil types. That can be facilitated by overbuilding the slopes and trimming them back to the compacted core (which is preferable to rolling the surface of the slope).

Stability fills should be keyed into firm underlying soil or competent bedrock. The key should be at least as wide as the stability fill and should extend at least 3 feet below the toe of the slope. Both the gross and surficial stability of the stability fill should meet the minimum stability requirements set by the governing agency. The gross or deep-seated stability should be analyzed along failure surfaces extending through the toe of the slope and beneath the keyway. Combinations of circular and non-circular failure surfaces should be used as applicable.

12.2.4 Buttress Fills

A buttress fill provides the features of a stability fill, but is used where a slope does not have a sufficient factor of safety for gross or deep-seated stability and additional resistive forces are required. For example, buttress fills can be used to support upslope landslides or slopes in sedimentary rock where the bedding is adversely dipping out of the slope.

The base of a buttress fill is typically wide, usually ranging from about one third to almost the full height of the slope being buttressed. The actual width of the buttress must be determined by slope stability analysis. Soil placed in the buttress fill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D1557, unless a different degree of compaction is recommended by a Geotechnical Engineer or required by the governing agency. Water content also should be controlled, as discussed in Section 12.2.3. Buttress fills should be keyed into competent underlying materials. The key should be at least as wide as the base of the buttress fill and should extend at least 3 feet below the toe of the slope. The required depth of the keyway must be evaluated by slope stability analysis. Benches should be cut into the native soil as the fill progresses to eliminate a planar interface at the fill/native soil contact. The vertical height of each bench typically should range from about 2½ to 5 feet.

A typical buttress is illustrated in Figure (9.1f). Failure surfaces that pass through and beneath a buttress fill must be analyzed. Combinations of circular and non-circular failure surfaces should be utilized. Typical critical failure paths that must be analyzed are surfaces extending through the toe of the buttress and base shear failures between the buttress and parent material. Typical modes of failure requiring analysis are depicted on Figure 9.1f. Both the gross and surficial stability of a buttress fill should meet the minimum stability requirements set by the governing agency.

12.2.5 Shear Keys

In some cases, the shear resistance of soil along a deep potential failure plane can be significantly increased by excavating a keyway into competent material below the potential failure surface and backfilling the keyway with compacted fill, slurry, or concrete. Stability analyses for slopes with a shear key should be performed using an appropriate shear strength for the keyway backfill material. Potential failure surfaces passing through and beneath the shear key should be considered.

12.2.6 Subdrains

Two types of subdrains can be used to maintain low water pressures within engineered slopes: backdrains and chimney drains. Backdrains are generally used behind stability fills, buttress fills, and beneath zones of total removal and replacement to maintain low water-pressures. Backdrains can consist of a 4-inch-diameter perforated or slotted pipe for small slopes or slopes where frequent outlets can be provided. Larger-diameter pipes may be required where significant quantities of water are anticipated or where the distance to an outlet point exceeds 200 feet. The lowest backdrain pipe should be placed along the backcut at the heel of the keyway and be as low as possible while still maintaining gravity flow to an outlet. Additional pipes should be located at 12- to 20-vertical foot intervals up the backcut. The backdrain pipe should be placed with the perforations down and be surrounded by 3 to 9 cubic feet of graded filter material per lineal foot of pipe. A solid pipe should collect water from the backdrain and discharge the water onto a non-erodible surface at the face of the slope. A backdrain pipe should not extend more than 200 feet without discharging into a collector pipe. The backdrain and outlet pipes should be sloped toward an outlet at about a 2-percent or steeper gradient.

Chimney drains can be provided every 25 to 50 linear feet at the interface of the stabilization fill and natural ground to enhance the backdrain system performances. The purpose of a chimney drain is to collect subsurface water from multiple bedding planes. The use of chimney drains is particularly important for buttress fills that will support bedded rock with considerably different permeability between layers. Conventional near-horizontal subdrains often will not collect water from the permeable layers because they do not intersect or cross the permeable beds. The chimney drains should be continuous between lateral backdrains and should be a minimum of 2 feet in width. Chimney drains may be created by stacking gravel-filled burlap (not woven plastic) bags, placement of a continuous gravel column surrounded by non-woven filter fabric, or placement of a drainage composite. Drain locations and outlet pipes should be surveyed in the field at the time of installation.

12.3 ENGINEERED STABILIZATION DEVICES AND SOIL IMPROVEMENT

A grading solution to a slope stability problem is not always feasible due to physical constraints such as property-line location, location of existing structures, the presence of steep slopes, and/or the presence of very low-strength soil. In such cases, it may be feasible to mechanically stabilize the slide mass or to improve the soil with admixture stabilization. The resulting slope should be analyzed to meet the same requirements as other slopes.

Mechanical stabilization of slopes can be accomplished using retaining walls, deep foundations (i.e., piles or drilled shafts), soil reinforcement with geosynthetics, tieback anchors, and soil nails. Common admixture stabilization measures include cement and lime treatment as well as Geofibers™.

12.3.1 Deep Foundations

The factor of safety of a slope can be increased by installing soldier piles/drilled shafts through the unstable soil into competent underlying materials. The piles/drilled shafts are sized and spaced so as to provide the required additional resisting force to achieve adequate slope stability. The piles/drilled shafts typically provide resistance through the bending capacity of the shaft anchored by passive resistance in stable earth materials underlying the slide mass.

The load applied to the deep foundation from material above the potential failure surface is commonly represented using a uniform or equivalent fluid pressure (triangular) distribution. Resistance to failure is provided by passive earth pressure within the "stable earth materials." In this context, stable earth materials are defined as those materials located beneath the potential failure surface having a static FS $\geq$ 1.5 and along which the anticipated seismic displacement is less than 5 cm or 15 cm (with the effects of the deep foundations and any other stabilization devices such as tieback anchors excluded in the analysis). In general, no resistance should be assumed above that failure surface, even though the failure surface with the minimum level of stability may be considerably higher/closer to the ground surface. An exception occurs when the wedge of soil downslope of the piles and above the surface with a static FS $\geq$ 1.5 possesses a factor of safety greater than 1.5 when analyzed as a free body or does not experience more than the allowable seismic displacement when analyzed as a free body. Passive pressures in those stable earth materials are strongly influenced by overburden pressures applied by the overlying slide mass. The effect of this overburden can be included if the material downslope of the deep foundations possesses adequate stability, but again should be neglected if this stability does not meet design requirements.

Analysis of the required resistance force to be provided by piles/drilled shafts is relatively straightforward when a single plane of weakness (i.e., a landslide slip surface) or a change in earth material (i.e., basal fill contact) defines the boundary between unstable and stable material. In those cases, the pressure applied by the earth material above the slide surface or contact generally is calculated assuming no lateral support from the same material downslope of the pile/drilled shaft. In some cases, the material above the slide surface/basal contact, located downslope of the foundations, can provide resistance in the form of active pressures. That resistance is only applicable when the stability of the wedge of material downslope or below the deep foundation exceeds design requirements.

Analysis of the resisting force to be provided by piles/drilled shafts is more complicated when a single failure surface or plane of weakness (i.e., unfavorably oriented bedded rock or a homogenous but relatively weak material) does not exist. In those cases, the engineer must first determine the lowest surface with the minimum required factor of safety (e.g. 1.5) or maximum allowable seismic displacement. That surface will be deeper than the surface yielding the lowest factor of safety or maximum seismic displacement. Determination of the required surface in bedded rocks is facilitated by the use of a program that allows the use of anisotropic strength parameters for different failure surface orientations. Once the lowest/deepest surface with the required minimum factor of safety allowable maximum seismic displacement is established, the pile/ drilled shaft load must be calculated. The method of analysis is the same as described above. Again, depending on the slope configuration relative to the pile/drilled shaft row, the material downslope of the pile/drilled shaft row may or may not provide lateral resistance.

The required embedment depth of the pile into stable bearing material should be determined by analyses considering the applied loads and resistance in stable earth materials.

Soldier piles/drilled shafts used to stabilize a slope may also be used to support other structures, provided the structures can tolerate the deflection that can be reasonably expected to occur. If the location of piles/drilled shafts relative to other engineered improvements is such that deflections of the deep foundations are of concern, deflections can be calculated based on soil properties evaluated using unfactored soil strengths. Soldier piles/drilled shafts used to stabilize the slope and provide support for a structure should be tied in two lateral directions such that the potential for lateral separation is minimized.

12.3.2 Tieback Anchors

The loads on the soldier piles/drilled shafts are, in some cases, higher than these elements can support in cantilever action alone. Tieback anchors can be incorporated in those cases to provide additional resistance. Tieback anchors also can be used without soldier piles/drilled shafts by anchoring them against a wall or reinforced face element. Tieback anchors consist of steel rods or cables that are installed in a drilled, angled holes. The rods/cables are grouted in place within the reaction zone and extend through a frictionless sleeve in the unstable mass. The anchors are post-tensioned after the grout reaches its design strength. Anchors are often tested to a load that is higher than the design load. The anchors must be long enough to extend into stable earth materials as defined in Section 12.3.1.

Temporary anchors generally do not need to be protected from corrosion. Permanent anchors should be protected from corrosion for the design life of the project. A reference for the design of ground anchors is Sabatini et al. (1999).

12.3.3 Soil Nails

Soil nailing involves earth reinforcement by placing and grouting reinforcing rods in holes drilled in the ground. The reinforcing rods are not pre-stressed or post-tensioned. Soil nailing should not be used in relatively fines-free gravel and sandy soil. A reference for the design of soil nails is Bryne et al. (1996). Soil nailing for permanent slope stabilization has been widely used by CalTrans and FHWA in Public Works projects. The application of this technique for general use is currently being studied by a special committee in southern California.

12.3.4 Retaining Structures

A retaining wall can be constructed through an unstable slope to provide additional resistance and raise the factor of safety for material behind the wall to an acceptable level. Retaining structures should be founded in stable earth materials as defined in Section 12.3.1. The retaining structure should be evaluated for possible sliding, overturning, and bearing failures using standard techniques. Failure surfaces that extend below the wall foundation and above the top of the wall also should be analyzed. Analysis of walls that support bedded rock dipping toward the wall is facilitated by use of a computer program that also allows the use of anisotropic strength parameters. Consideration must be given to whether material in front of the wall that is assumed to provide passive resistance could be removed or excavated in the future. In some cases, the retaining wall system may consist of tiebacks and soldier piles/drilled shafts.

12.3.5 Strengthened or Reinforced Soil

The strength characteristics of compacted fill can be improved by mixing the soil with cement or lime during compaction or by mechanically reinforcing the soil. In the case of admixture stabilization, testing is required to determine the type and amount of admixture necessary to achieve the required strength. Soil with more than 50 percent fines (passing the #200 sieve) is not well suited for mixing with cement. Moist fine-grained soil is often suitable (amenable to) for lime treatment. Winterkorn and Pamukcu (1991) provide a reference on admixture stabilization.

Soil reinforcement is commonly accomplished with geosynthetics such as woven geotextiles, geogrids, or steel strips. The reinforcement should extend beyond the failure surface that has a minimum factor of safety of 1.5 and the allowable seismic displacement. A reference on the application of those materials is provided by Koerner (1998).

12.4 DEWATERING

The presence of water in a slope can reduce the shear strength of the soil, reduce the shear resistance through buoyancy effects, and impose seepage forces. Those effects reduce the factor of safety of the slope and can cause failure of the slope. Dewatering a slope (removing subsurface water) and/or providing drainage control to prevent future subsurface water build-up can increase the factor of safety. Both passive and active dewatering/subsurface-water-control systems can be used. Many dewatering systems require periodic maintenance to remain effective. In addition, monitoring programs may be required to document or verify the effectiveness of the system.

A slope can be "passively" dewatered by installing slightly inclined gravity dewatering wells into the slope. Those "horizontal" drains (also known as hydraugers) should be sloped toward an outlet and extended sufficiently into the slope to dewater the earth materials that affect the stability of the slope. Vertical pumped-wells also can be utilized to lower subsurface water levels within a potentially unstable mass.

The effectiveness of dewatering wells is dependent on the permeability of the soil. In some cases, the soil is not sufficiently permeable, or other conditions exist that preclude effective dewatering of the slope.

The effectiveness of dewatering drains or wells needs to be checked periodically by measuring the water levels in the slope. Drains and wells, whether pumped or static, require periodic maintenance to assure that the casing does not become clogged by fines or precipitates and that the pump is functioning. The effectiveness of subsurface drainage control features is dependent on proper maintenance of the drains and/or wells. Where proper maintenance of the wells/drains cannot be guaranteed for the time period during which the stability of the slope is to be maintained, a dewatering system should not be relied upon to achieve the required factor of safety.

"Passive" dewatering with subdrains was discussed previously in section 12.2.6.

12.5 CONTAINMENT

Loose materials, such as colluvium, slopewash, slide debris, and broken rock, on the slope that could pose a hazard can be collected by a containment structure capable of holding the volume of material that is expected to fail and reach the containment device over a given period of time. The containment structure type, size, and configuration will depend on the anticipated volume to be retained and the configuration of the site. Debris basins, graded berms, graded ditches, debris walls, and slough walls can be used. In some cases, debris fences may be permitted, although those structures often fail upon high-velocity impact.

The expected volume of debris should be estimated by the geologist and engineer. Debris walls and slough walls should be designed for a lateral equivalent pressure of at least 125 pounds per cubic foot where impact loading is anticipated and at least 90 pounds per cubic foot elsewhere unless otherwise allowed by the regulatory agency and/or justified by the consultant. The height of the catchment devices may be governed by the expected debris volume of the expected bounce height of a rolling rock. The CRSP program (Jones, et al., 2000) can be used to estimate rolling rock trajectories.

Access should be provided to debris containment devices for maintenance. The type of access required is dependent on the anticipated volume of debris requiring removal. Wheelbarrow access will be sufficient in some cases, whereas heavy equipment access may be required in other areas.

12.6 DEFLECTION

Walls or berms that are constructed at an angle to the expected path of a debris flow can be used to deflect and transport debris around a structure. The channel gradient behind those walls or berms must be sufficient to cause the debris to flow rather than collect. Required channel gradients may range from 10 to 40 percent depending on the expected viscosity of the debris and whether the channel is earthen or paved. An area for debris collection should be provided at the terminus of the deflection device.

12.7 SLOPE PROTECTION FOR ROCK SLOPES

Woven wire mesh and wire mesh have been used to mitigate rock fall hazards. The mesh is hung from anchors drilled into stable rock and is placed over the slope face to help keep dislodged rocks from bouncing as they fall. The bottom of the mesh is generally left open so that dislodged rocks do not accumulate behind the mesh and cause it to fail. Usually a ditch is provided at the toe of the slope to collect fallen rock. Wire mesh systems can contain large rocks (3 feet in diameter) traveling at fast speeds. It is also possible to hold rocks in position with cables, rock bolts, or gunite slope covering.

12.8 RESISTANT STRUCTURES

Structures can sometimes be designed to resist damage during the anticipated slope movement. Examples of structural systems that can resist damage include mat foundations and very stiff, widely spaced piles. Mat foundations are designed to resist or minimize deflection or distortion of the structure resting on the mat as a result of permanent displacement of the underlying ground. The mat foundation itself may move or settle differentially, but the mat is sufficiently stiff to reduce bending in the structure to a tolerable level. Mat foundations can be particularly useful when compacted fill slopes are expected to experience greater than 5 cm of seismic displacement in the area of a habitable structure. It must be recognized, however, that permanent vertical differential settlement may be undesirable and releveling may be required after the design event.

Another instance where a building can be designed to resist damage to earth movement involves structures built over landslides experiencing plastic flow. Landslides that do not move as a rigid block can be penetrated with a series of widely spaced stiff piles. These piles are designed to resist loading imposed by material acting on some tributary area to the piles (generally wider than the pile). The remaining material is designed to flow between the piles. The access and utilities leading to the building must be designed assuming that the ground surface will move vertically and laterally relative to the structure.

13 CONCLUDING REMARKS

This document has presented a broad overview of landslide hazard analysis, evaluation, and mitigation techniques. The Implementation Committee acknowledges that the state of the art in slope stability evaluation continues to evolve and advance and that new methodologies in geotechnical engineering, soil/shear strength testing, slope-stability analysis, and mitigation will develop.

Many of the issues germane to this topic, such as strength evaluation and the treatment of uncertainties, were the subjects of extended debate by the Committee. Typically at issue was the pervasive use in current practice of antiquated technologies that provide misleading, or at best highly uncertain, outcomes. All too often, the Committee was compelled to adopt language encouraging (or at least allowing) the use of such technologies when more robust (but invariably more expensive) alternatives exist. One important example of this is the use of direct shear strength testing of samples from Modified California samplers. Another is the continued use of a static FS=1.5 regardless of the level of subsurface characterization and project importance. Technologies currently exist, and continue to be developed, that allow geotechnical engineering practice to move beyond gross conservatism and almost purely judgment-based design. What is needed is clear recognition by consultants, regulators, and owners of the economic and societal benefits of proper geotechnical work. If the provisions in this document are adopted in practice, it will represent a small step in the right direction, but all parties involved must remain diligent in trying to advance the all too often tradition-bound profession we share.

The implementation of SP 117 represents an important step in furthering seismic safety in the State of California. Proper analysis of both the static and seismic stability of slopes is critical to the safety and well being of Californians as development continues to expand into hillside areas. It is the hope of the Implementation Committee that this document will make a contribution toward that goal and provide useful information and guidance to owners, developers, engineers, and regulators in the understanding and solution of the slope stability and landslide hazards that exist in California and in other tectonically active regions.

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APPENDIX A

DEVELOPMENT OF A SCREEN ANALYSIS PROCEDURE FOR SEISMIC SLOPE STABILITY

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ABSTRACT

Site-specific seismic slope stability analyses are required in California by the 1990 California Seismic Hazards Mapping Act for sites located within mapped hazard zones and scheduled for development with more than four single-family dwellings. A screen analysis is performed to distinguish sites for which only small ground deformations are likely from sites for which larger, more damaging landslide movements could occur. No additional analyses are required for sites that pass the screen, whereas relatively detailed analyses are required for sites that fail the screen. We present a screen analysis procedure that is based on a calibrated pseudo-static representation of seismic slope stability. The novel feature of the present screen procedure is that it accounts not only for the effects of ground motion amplitude on slope displacement, but also accounts for duration effects indirectly via the site seismicity. This formulation enables a more site-specific screen analysis than previous formulations that made *a priori* assumptions of seismicity/duration.

INTRODUCTION

The 1990 California Seismic Hazards Mapping Act called upon the California Division of Mines and Geology (CDMG) to map geographic areas considered to be potentially susceptible to earthquake-induced liquefaction or landslides. For developments located in these "Special Studies Zones" that include more than four single-family dwellings, engineers must perform site-specific studies to evaluate whether the mapped hazard actually exists. If a hazard is identified, appropriate remedial measures must be taken.

Working with the CDMG, a number of southern California municipal and county agencies formed committees of experts charged with developing detailed guidelines for implementation of the Hazard Act's liquefaction and landslide components. The liquefaction guidelines (Martin and Lew, 1999) largely follow the recommendations developed by a separate international committee of experts (Youd et al., 2001). No such consensus document exists for seismic slope stability, however, so the Landslide Hazards Implementation Committee (i.e., the "Committee") has developed over the course of three years an original guidelines document (i.e., the main body of this report). This guidelines document addresses a broad suite of issues, including drilling and sampling techniques, shear strength evaluation, evaluation of static slope stability, evaluation of seismic slope stability, and mitigation of slope stability hazards. For the most part, the Committee drew upon existing research and experience to draft the guidelines on these topics. The topic of seismic slope stability, however, proved to be particularly vexing and required the Committee to conduct limited new research so that guidelines appropriate for hillside construction could be developed. There are two principal components to the guidelines on seismic slope stability – a screen analysis to determine if a seismic stability hazard is likely to exist at the site, and a formal displacement analysis for sites that fail the screen. The objective of this appendix is to document the screen analysis procedure developed by the Committee and the process by which it was formulated.

EXISTING SCREEN PROCEDURES FOR SEISMIC SLOPE STABILITY

Screen analysis procedures for seismic slope stability have been adopted by a number of U.S. agencies with jurisdiction over hillside residential construction, earth dams, and solid-waste landfills. These procedures generally utilize a pseudo-static representation of seismic demand in which a de-stabilizing horizontal seismic coefficient (k) is utilized within a conventional limit equilibrium slope stability calculation. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the slide mass. The factor of safety against shear failure is checked with the equivalent horizontal force applied to the slope, and the slope passes the screen if the factor of safety exceeds a specified minimum value. For the sake of convenience, two types of seismic coefficients are introduced here for later reference. The first is the seismic coefficient that reduces the pseudo-static factor of safety (FS) for a given slope to unity, and is referred to as the yield acceleration, k_y. The second is the peak value of spatially averaged horizontal acceleration (normalized by g) across the slide mass, and is denoted k_{max}.

Perhaps the most widely used screen analysis procedure is that developed by Seed (1979) for application to earth dams. The procedure calls for k = 0.1 or 0.15 to be applied for M = 6.5 and 8.25 earthquakes, respectively. The screen is passed if the factor of safety, FS, exceeds 1.15. A slightly modified version of that procedure, in which k = 0.15 and FS ≥ 1.1 regardless of local seismicity, was adopted in 1978 by Los Angeles County for application to hillside residential construction. Seed (1979) recommended that his procedure only be applied for cases where the earth materials do not undergo significant strength loss upon cyclic loading (i.e., strength loss < 15%) and where several feet of crest displacement was deemed "acceptable performance," as is the case for many earth dams (e.g., 0.9 m displacement for M = 8.25 and crest acceleration = 0.75g).

An important feature of the Seed (1979) procedure is its calibration to a particular slope performance level, which is represented by the displacement of a rigid block on an inclined plane (i.e., a "Newmark-type" displacement analysis, Newmark, 1965). Seed (1979) calibrated his pseudo-static approach using Newmark displacements calculated with simplified methods (e.g., Makdisi and Seed, 1978). The Makdisi and Seed simplified procedure, in turn, is based on a limited number of calculations that were used to relate Newmark displacement to earthquake magnitude and k_y/k_{max} (e.g., five calculations for M = 6.5, two for M = 7.5, and two for M = 8.25). Seed's (1979) recommendations are an important milestone, as they represent the first calibration of a pseudo-static method to a particular level of slope performance as indexed by displacement. This concept underlies other widely used screen analysis procedures that have been developed to date, and is retained as well in the present work.

Since the Seed (1979) work, additional screen analysis procedures have been developed for application to earth dams and solid waste landfills. A procedure for earth dams was developed by Hynes-Griffin and Franklin (1984) based on (1) calculations of shaking within embankment sections using a linear elastic shear beam model by Sarma (1979) and (2) calculations of Newmark displacement from time histories using the analysis approach of Franklin and Chang (1977). Those calculations resulted in statistical relationships between the amplification of shaking within embankments (i.e., ratio of $k_{max} \times$ g to maximum horizontal acceleration of base rock, MHA_r) and the depth of the sliding surface, as well as between Newmark displacement and k_y/k_{max}. Hynes-Griffin and Franklin (1984) developed their pseudo-static procedure using approximately a 95th percentile value of amplification for deep sliding-surfaces along with the upper-bound value of k_y/k_{max} that produces 1.0 m of displacement. In the resulting procedure, k is taken as $0.5 \times MHA_r$, and the screen is passed if FS ≥ 1.0. The procedure is intended for use with 80% of the shear strength in non-degrading materials. The method is not recommended for areas subject to large earthquakes, embankments constructed of or on liquefiable soil, or embankments for which small displacements are intolerable.

Bray et al. (1998) used a similar procedure to that of Hynes-Griffin and Franklin (1984) to develop a screen procedure for solid-waste landfills. As with the earlier procedure, two suites of statistical results underlie the procedure. One relates the peak acceleration of the slide mass ($k_{max} \times$ g) to MHA_r, the other relates displacement for a given k_y/k_{max} to the amplitude and duration of shaking. A large number of calculations were performed by Bray et al. (1998) to establish these relationships, which are discussed in more detail below. The screen procedure was developed using nearly upper bound amplification factors (i.e., $k_{max} \times g/MHA_r$) and tolerable displacements of about 0.15-0.3 m. The resulting procedure calls for k to be taken as $0.75 \times MHA_r$, and the screen is passed if k > k_y (which is analogous to having FS ≥ 1 when k is applied in a pseudo-static analysis).

The above is not a comprehensive review of all screen procedures developed to date for seismic slope stability. Rather, our intent is to illustrate the principal steps taken in the development of commonly used, rational screen procedures, and the conditions for which those procedures are intended to be applicable. Three important conditions underlie the procedures: (1) the level of displacement considered tolerable for a specific application, (2) the earthquake magnitude associated with the time histories used to calculate displacements, and (3) the level of conservatism employed in the interpretation of statistical distributions of results. Discussion on those three points is provided below:

- The limiting displacements used by Seed (1979) and Hynes-Griffin and Franklin (1984) for earth dams were on the order of 100 cm. The limiting displacements used by Bray et al. (1998) for landfills were 15-30 cm.

- The earthquake magnitude used by Seed (1979) in developing the criteria subsequently adopted by L.A. County is 8.25. The time histories used by Hynes-Griffin and Franklin (1984) are from magnitudes that range from 3.8 to 7.7, with most being 6.6 (San Fernando earthquake). Bray et al. (1998) did not use magnitude directly, but instead used duration, which is strongly correlated to magnitude. The durations used by Bray et al. are consistent with earthquake magnitudes of about 7 to 8, with most being closer to 8 (J. Bray, pers. communication).

- Seed (1979) exercised conservatism by using upper-bound values of displacement for a given k_y/k_{max}. Hynes-Griffin and Franklin (1984) were highly conservative through their use of 95th percentile amplification levels coupled with upper-bound displacements for a given k_y/k_{max}. Bray et al. (1998) were also conservative with their use of nearly upper-bound amplification levels and 84th percentile displacements.

The screen analysis procedure developed herein is intended principally for application to hillside residential and commercial developments. For construction of this type, small ground deformations can cause collateral loss that is considered unacceptable by owners, insurers, and regulatory agencies. Accordingly, the limiting displacements used in existing screen procedures for earth dams and landfills are considered to be too large for application to hillside construction. Another problem with the existing procedures is the level of conservatism employed in their development. For example, the existing methods apply for specific ranges of earthquake magnitude (which are high for the Seed and Bray et al. methods), and may not pass otherwise safe sites for which the design magnitude is smaller than that used in the development of the screen. Moreover, the conservative interpretation of amplification and displacement distributions used in the development of existing schemes likely makes the level of risk associated with the slope performance differ significantly from that associated with the ground motions. In other words, if the ground motion is evaluated with probabilistic hazard analysis for a given return period, and the slope displacement conditioned on that ground motion is extreme (i.e., a rare realization), the resulting slope design is based on displacements having a much longer return period than the design-basis ground motion.

Given those shortcomings, the Committee has developed a new screen procedure tailored to the needs of hillside residential and commercial construction (in terms of displacement) and which accounts for site-specific seismicity. The screen procedure was also developed so as to control the level of conservatism in order to maintain a reasonable return period on the expected slope performance. The remainder of this appendix describes the development of the procedure.

DEVELOPMENT OF SCREEN ANALYSIS PROCEDURE

Introduction

The purpose of screen investigations for sites within zones of required study is to filter out sites that have no potential or low potential for earthquake-induced landslide development. No additional seismic stability analysis is required for a site that passes the screen, whereas further quantitative evaluation of landslide hazard potential (and possibly mitigation) is required for sites that fail the screen.

Like other screen procedures described in the previous section, ours is based on a pseudo-static representation of seismic slope stability. The procedure is implemented by entering a de-stabilizing horizontal seismic coefficient (k) into a conventional slope stability analysis. The seismic coefficient represents the fraction of the weight of the sliding mass that is applied as an equivalent horizontal force acting through the centroid of the mass. If the factor of safety is greater than one (FS > 1), the site passes the screen, and the site fails if FS < 1.

We formulate the seismic coefficient as the product of the maximum horizontal acceleration at the site for a rock site condition (MHA_r) and a factor (f_{eq}) related to the seismicity of the site, the maximum tolerable slope displacement, and other factors:

$$k = f_{eq} \times MHA_r / g \tag{1}$$

where g is the acceleration of gravity. The two key steps in the development of the screen procedure are therefore (1) rationale for the use of MHA_r to represent the amplitude of shaking within the slide mass, and (2) formulation of f_{eq} to represent the effects of local seismicity and the maximum tolerable slope displacement. The following two subsections discuss these steps.

Amplitude of Shaking in Slide Mass

Ideally, the MHA_r/g term in Eq. 1 should represent the spatially averaged normalized amplitude of shaking within the slide mass, which differs from the maximum horizontal acceleration at the base of the slide for a rock site condition (MHA_r) as a result of ground response and topographic effects within the slide mass (which can amplify or de-amplify shaking) and vertical and lateral incoherence of ground motion within the slide mass (which tends to de-amplify shaking). Bray et al. (1998) define the spatially averaged peak acceleration of a slide mass as the maximum horizontal equivalent acceleration (MHEA). Parameter MHEA is a more direct indicator of shaking amplitude in a slide mass than MHA_r and hence Eq. 1 could be re-written as,

$$k = f_{eq}^{*} \times MHEA / g \tag{2}$$

where $f_{eq}^* = f_{eq} \times (MHA_r/MHEA)$. Bray et al. (1998) evaluated MHEA as a function of MHA_r from calculations of wave propagation through an equivalent one-dimensional slide mass. As shown in Figure 1, Bray et al. normalize calculated MHEA in the slide mass by the product of MHA_r and a nonlinear response factor (NRF), which accounts for nonlinear ground response effects as vertically propagating shear waves pass through the slide mass. Bray et al. use MHA_r as the normalizing ground motion even for sites where the foundation materials are soil because their analyses did not indicate site condition as significantly affecting MHEA (except for deep soft clay sites, for which site specific analyses were recommended). The ratio $MHEA/(MHA_r \times NRF)$ differs from one as a result of vertical ground motion incoherence within the slide mass, and is related in Figure 1 to the ratio of the period of the sliding mass (T_s) to the mean period of the input motion (T_m). The ratio $MHEA/(MHA_r \times NRF)$ is less than one for $T_s/T_m > \sim 0.5$, and is variable with an average of about 1.0 for $T_s/T_m < \sim 0.5$.

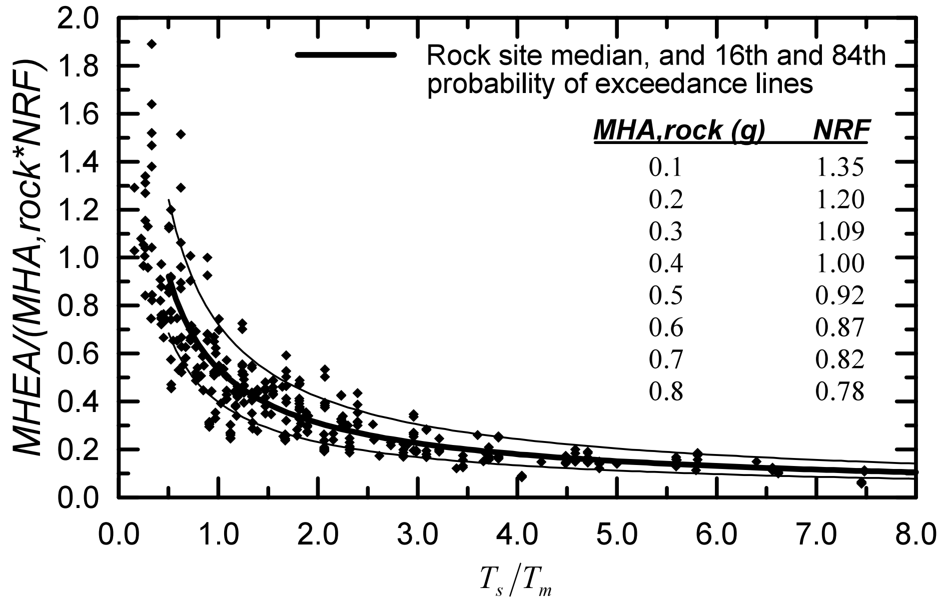

Fig. 1. Normalized MHEA for Deep-Seated Slide Surface vs. Normalized Fundamental Period of Slide Mass (after Bray et al., 1998).

The magnitude and distance that control the peak acceleration hazard in much of urban southern California are magnitude 6.5 – 7.0 earthquakes at distances generally less than 10 km (Petersen et al., 1996). Parameter T_m has a median value of about 0.5 s for these magnitude and distance ranges (Rathje et al., 1998). Parameter T_s is calculated as

$$T_s = \frac{4H}{V_s} \tag{3}$$

where H = thickness of slide mass and V_s = average shear wave velocity of slide mass. If V_s is taken as 300 m/s (consistent with soft bedrock or compacted fill materials), the slide mass thickness would have to exceed about 20 m for $T_s/T_m > 0.5$. It was therefore the Committee's judgment that MHEA/($MHA_r \times$ NRF) = 1.0 would be a reasonable assumption for sites having critical slip surfaces of moderate to shallow depth (< ~20 m), and would be conservative for deeper-seated slip surfaces (depth > ~20 m). Because parameter NRF is a function of MHA_r (as shown in Figure 1) the assumption of MHEA/($MHA_r \times$ NRF) = 1.0 makes MHEA solely a function of MHA_r. Accordingly, Eq. 2 can be re-written as Eq. 1 provided the effect of NRF is incorporated into factor f_{eq}, which is done in the next section.

Formulation of Seismicity Factor f_{eq}

For a given MHA_r, large magnitude earthquakes will tend to cause poorer slope performance than smaller magnitude earthquakes. One important reason for this is that large magnitude earthquakes have longer durations of shaking. Previous pseudo-static procedures for seismic slope stability have specified a single value for f_{eq}, and thus have made implicit, and usually very conservative, assumptions about the magnitude of earthquakes causing the design-basis MHA_r. The Committee sought to reduce that conservatism by developing a range of f_{eq} values that are a function of magnitude and site-source distance.

Magnitude- and distance-dependent f_{eq} values were developed using a statistical model that relates slope displacements from a Newmark-type analysis (u) to the amplitude of shaking in the slide mass (k_{max} = MHEA/g), significant duration of shaking (measured as the time between 5-95% normalized Arias intensity, $D_{5\text{-}95}$), and the ratio k_y/k_{max} (where k_y = horizontal seismic coefficient that reduces the factor of safety for the slope to unity). The statistical model employed here was developed by Bray and Rathje (1998) from regression analysis of 309 Newmark-displacement values at each of four k_y/k_{max} ratios. The model and data from Bray and Rathje are shown in Figure 2, and indicate a log-normal distribution of normalized displacement $u/(k_{max} \cdot D_{5\text{-}95})$ for a given k_y/k_{max} ratio. Regression analyses indicate that the median of this log-normal distribution is described by,

$$\log_{10}\left(\frac{u}{k_{\max} \cdot D_{5-95}}\right) = 1.87 - 3.477 \cdot \frac{k_y}{k_{\max}} \tag{4}$$

where u is the median displacement in cm. The standard error is 0.35 in $\log_{10}$ units.

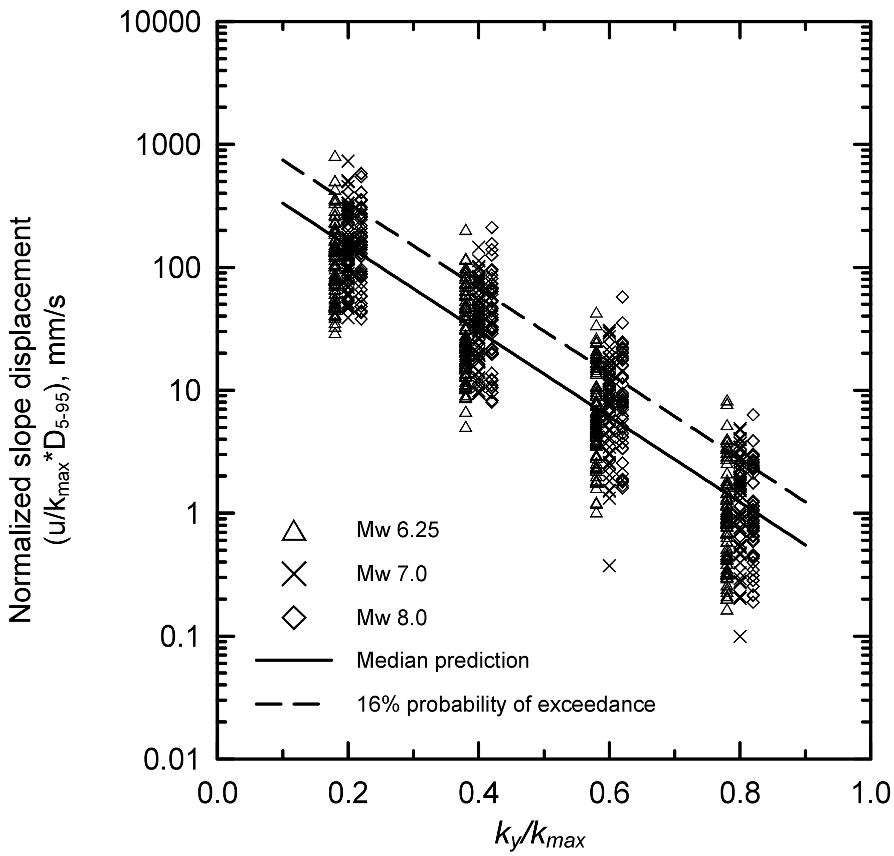

Fig. 2. Normalized Sliding Displacement (modified from Bray and Rathje, 1998).

A relationship between magnitude, distance, MHA_r, and f_{eq} was established using the Bray and Rathje relationship with the following assumptions and observations:

1. Factor f_{eq}^* (Eq. 2) was taken as equivalent to k_y/k_{max}. The equivalency of k_y/k_{max} and f_{eq}^* can be understood by recognizing that k_y/k_{max} simply represents the factor by which the actual ground shaking intensity (k_{max}) needs to be reduced to render a seismic coefficient associated with FS = 1 (i.e., $k_y = k_y/k_{max} \times k_{max}$). Referring to Eq. 2, because our screen procedure is intended for use with FS = 1, f_{eq}^* represents the factor by which MHEA/g needs to be reduced to yield a seismic coefficient associated with FS = 1 (i.e., k_y). Accordingly, if k_y is substituted for k in Eq. 2 (appropriate for FS = 1) and k_{max} is substituted for MHEA/g, it can be readily seen that $f_{eq}^* = k_y/k_{max}$.

2. Parameter MHEA is inconvenient for use in a screen procedure because its relationship to MHA_r is affected by vertical ground motion incoherence effects and nonlinear ground response effects. As described in the previous section, to simplify the analysis we neglect the vertical incoherence effects, which is equivalent to assuming MHEA/(MHA_r × NRF) = 1.0. From Eq. 1 and 2, we see that $f_{eq} = f_{eq}^* \times MHEA/MHA_r$, which reduces to $f_{eq}^* \times$ NRF with the above assumption. Since $f_{eq}^* = k_y/k_{max}$, we calculate parameter $f_{eq} = k_y/k_{max} \times$ NRF.

3. Two threshold levels of Newmark displacement were selected by the Committee, u=5 and 15 cm. It should be noted that the Newmark displacement parameter is merely an index of slope performance. The 5 cm threshold value likely distinguishes conditions for which very little displacement is likely from conditions for which moderate or higher displacements are likely. The 15 cm value likely distinguishes conditions in which small to moderate displacement are likely from conditions where large displacements are likely. It should be noted that those threshold displacement values are smaller than values used in the development of existing screen procedures for dams and landfills. The Committee's use of the small displacement value is driven by a concern on the part of owners, insurers, and regulatory agencies to minimize collateral loss from slope deformations in future earthquakes.

4. Factor k_{max} is taken as MHA_r × NRF/g. Parameter D5-95 is a function of magnitude and distance, and can be estimated from available attenuation relationships.

Based on the above, calculations were performed to evaluate as a function of f_{eq} the probability that seismic slope displacement u > 5 cm conditional on MHA_r, magnitude, and distance. This probability is calculated as:

$$P(u > 5cm \mid MHA_r, M, r, f_{eq}) = \int_{D_{5-95}} f(D_{5-95} \mid m, r) P(u > 5cm \mid D_{5-95}(M, r), MHA_r, f_{eq}) d(D_{5-95}) \tag{5}$$

where $d(D_{5\text{-}95})$ represents a differential duration; $f(D_{5\text{-}95}|M,r)$ represents a log-normal probability density function described by the median and standard error calculated using an attenuation relationship for duration; the probability term is evaluated from the cumulative distribution function described by the median and standard error terms developed by Bray and Rathje (1998); and the integration is performed over a reasonable range of durations (taken as the median ± 2.5 standard deviations of duration for the given M and r). Similar calculations were performed for $u > 15$ cm.

To illustrate the application of Eq. 5, Figure 3 shows for $M = 7$, $r = 20$ km, $MHA_r = 0.4g$ and $u = 5$ cm the variation of the probability term on the left-hand side of Eq. 5 with f_{eq}. The distribution in Figure 3 is unity minus a normal cumulative distribution function with median 0.56 and standard error 0.117. The standard error term is related to the dispersion of the duration attenuation model and the Bray and Rathje displacement model, and is independent of M, r, MHA_r, and u. The Committee evaluated median f_{eq} values for a range of MHA_r, M, and r (e.g., 0.56 for the example in Figure 3) and for $u = 5$ cm and 15 cm. We chose to use the median because our judgment is that probabilities departing significantly from the 50th percentile would unnecessarily bias the effective return period for exceedance of the specified level of slope displacement (i.e., $u = 5$ cm) from the return period for the ground motion (typically 475 years). However, f_{eq} values for other percentiles can be readily evaluated from the median because the standard deviation is fixed at 0.117.

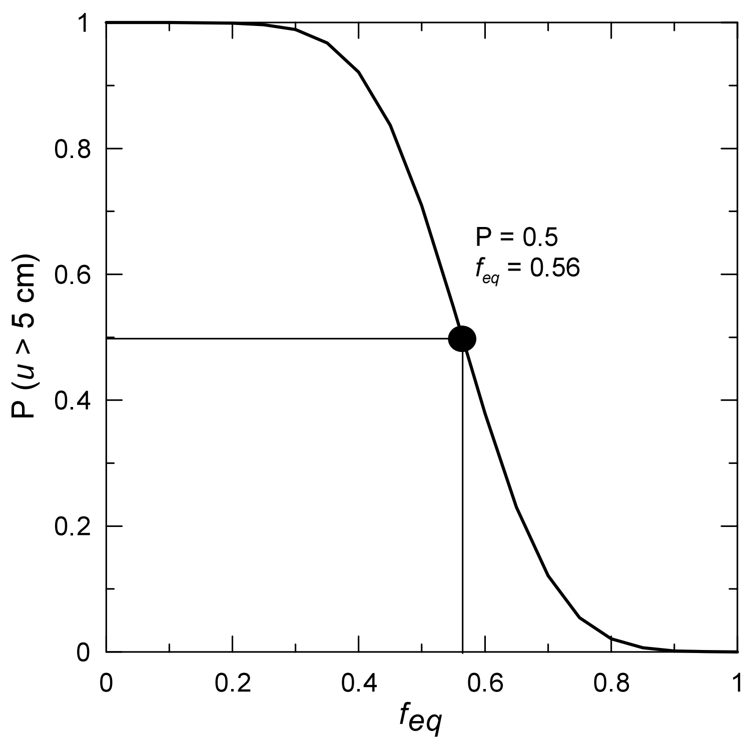

Fig. 3. Variation of Exceedance Probability for 5 cm Slope Displacement with f_{eq} for M = 7, r = 20 km, and MHA_r = 0.4g

The distribution of median f_{eq} values with M, r, and MHA_r are shown in Figure 4(a) for u = 5 cm and in Figure 4(b) for u = 15 cm. The values in Figures 4 were derived using the Abrahamson and Silva (1996) attenuation model for duration at rock sites. Near-fault effects on ground motion parameters were neglected in the development of Figures 4; such effects would tend to increase the amplitude of long-period components of the ground motion but decrease the duration, and hence the net effect on seismic slope displacements would likely be small. Focal mechanism does not affect these calculations because the Abrahamson and Silva attenuation model for duration does not contain a focal mechanism term.

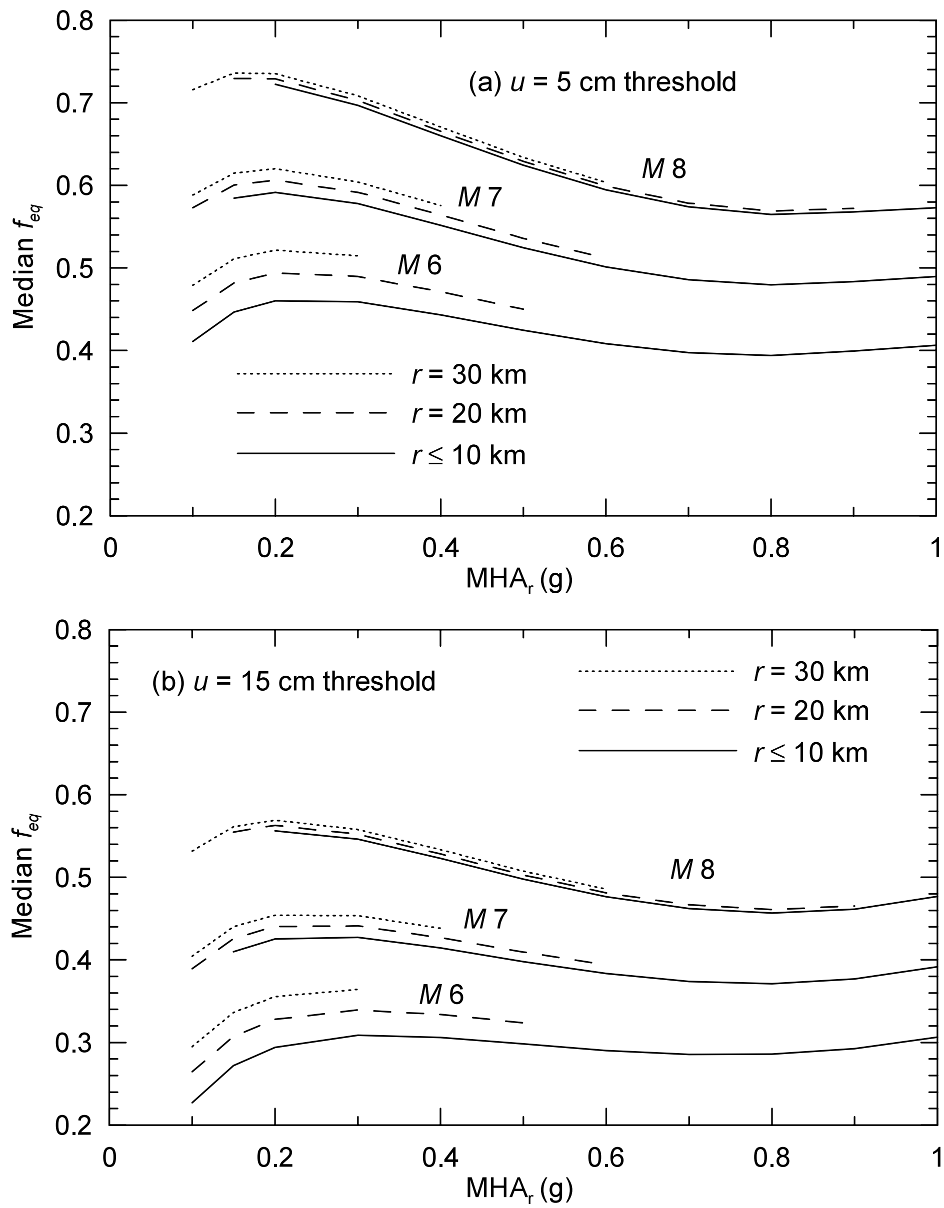

Fig. 4. Required Values of f_{eq} as Function of MHA_r and Seismological Condition for Acceptable Slope Performance

The equation of the curves in Figures 4 is as follows:

$$f_{eq} = \frac{NRF}{3.477} \times \left[1.87 - \log_{10}\left(\frac{u}{(MHA_r / g) \times NRF \times D_{5-95,m}} \right) \right] \tag{6}$$

where u = 5 or 15 cm, $D_{5\text{-}95,m}$ = median duration from Abrahamson and Silva (1996) relationship, defined by,

$$r > 10 \text{ km: } \ln(D_{5-95,m}) = \ln\left[\frac{\left(\frac{\exp[5.204 + 0.851 \cdot (M-6)]}{10^{1.5M+16.05}} \right)^{-1/3}}{15.7 * 10^6} + 0.063 \cdot (r-10) \right] + 0.8664 \tag{7a}$$

$$r < 10 \text{ km: } \ln(D_{5-95,m}) = \ln\left[\frac{\left(\frac{\exp[5.204 + 0.851 \cdot (M-6)]}{10^{1.5M+16.05}} \right)^{-1/3}}{15.7 * 10^6} \right] + 0.8664 \tag{7b}$$

and NRF is defined by the relationship tabulated in Figure 1, which can be approximated by:

$$NRF \approx 0.622 + 0.920 \exp(-2.25 \times MHA_r / g) \tag{8}$$

for $0.1 < MHA_r/g < 0.8$.

Referring to Figure 4, the strong increase in f_{eq} with magnitude and small increase with distance are driven by the duration attenuation model, which shows similar variations in $D_{5\text{-}95}$ with magnitude and distance. The variation with MHA_r is driven by the statistical displacement model (Eq. 4) and the NRF parameter. Without the NRF parameter, the curves in Figure 4 would increase linearly with the logarithm of MHA_r. Inclusion of the NRF parameter increases f_{eq} at small MHA_r and decreases f_{eq} at large MHA_r to the extent that f_{eq} is only weakly dependent on MHA_r.

As noted previously, f_{eq} values for percentiles other than 50 (i.e., the median) can be evaluated through use of the fixed standard error term of 0.117. For example, the 84th percentile values can be obtained by adding 0.117 to the f_{eq} values estimated from Eq. 6.

APPLICATION

Design Earthquakes

A critical issue associated with the application of the above screen procedure is selection of appropriate design-basis earthquakes. The Committee recommends that the MHA_r having a 475-year return period (10% probability of exceedance in 50 years) be estimated using probabilistic seismic hazard analysis (PSHA). The relative contributions of earthquake events at different magnitudes and distances to this MHA_r hazard should then be evaluated through a de-aggregation analysis, and the mode magnitude ($\overline{M}$) and mode distance ($\bar{r}$) identified for use in the screen. That combination of MHA_r, $\overline{M}$, and $\bar{r}$ represents the parameters that should be used to evaluate *k*. The Committee considered the use of supplemental deterministic seismic hazard analyses for sites located near large-magnitude, high slip-rate faults (such as the San Andreas fault system). However, it was found for many checked locations that *k* values computed deterministically were less than *k* values evaluated from PSHA. The PSHA results used in those checks are from published State-wide maps (Petersen et al., 1996). In our checks, the deterministic *k* values were evaluated using the characteristic earthquake event (as compiled by Petersen et al., 1996) on the largest fault segment nearest the site, and the 84^{th} percentile MHA_r value associated with that characteristic event. The Committee recognizes that more severe deterministic scenario events could be conceived, but those would likely be sufficiently rare as to have a return period that significantly exceeds the 475-year target.

Limitations

As with other screen analysis procedures, the present procedure should not be used for slopes comprised of geologic materials that could be subject to significant strain softening, such as liquefiable soil. The procedure is not applicable to slopes constructed over soft clay soil, because as noted previously the Bray et al. (1998) relationship for MHEA (Figure 1) does not apply for that site condition. The procedure also should not be applied to situations for which 5 cm (or 15 cm) displacement is an inappropriate displacement threshold. Finally, it should be noted that this screen analysis procedure, and any analysis of seismic slope stability based on Newmark sliding block models, only provides an index of slope performance that is related to the accumulation of permanent shear deformations within the ground. Volumetric ground deformations associated with post-liquefaction pore-pressure dissipation or seismic compression of unsaturated ground are not considered in Newmark-type models and need to be evaluated separately.

Examples

Seismic coefficients (*k*) for three example sites in southern California are evaluated to illustrate application of the screen procedure defined by Eqs. 1 and 6. Locations of the sites are shown in Figure 5. The site denoted "Los Angeles" in Figure 5 is on the north flank of the Santa Monica Mountains, and is not immediately adjacent to any major active fault systems. The site denoted "Glendale" is near the base of the San Gabriel Mountains, and is close to the Sierra Madre fault system. The site at the intersection of Highway 138 and Interstate Highway 5 is adjacent to the San Andreas fault.

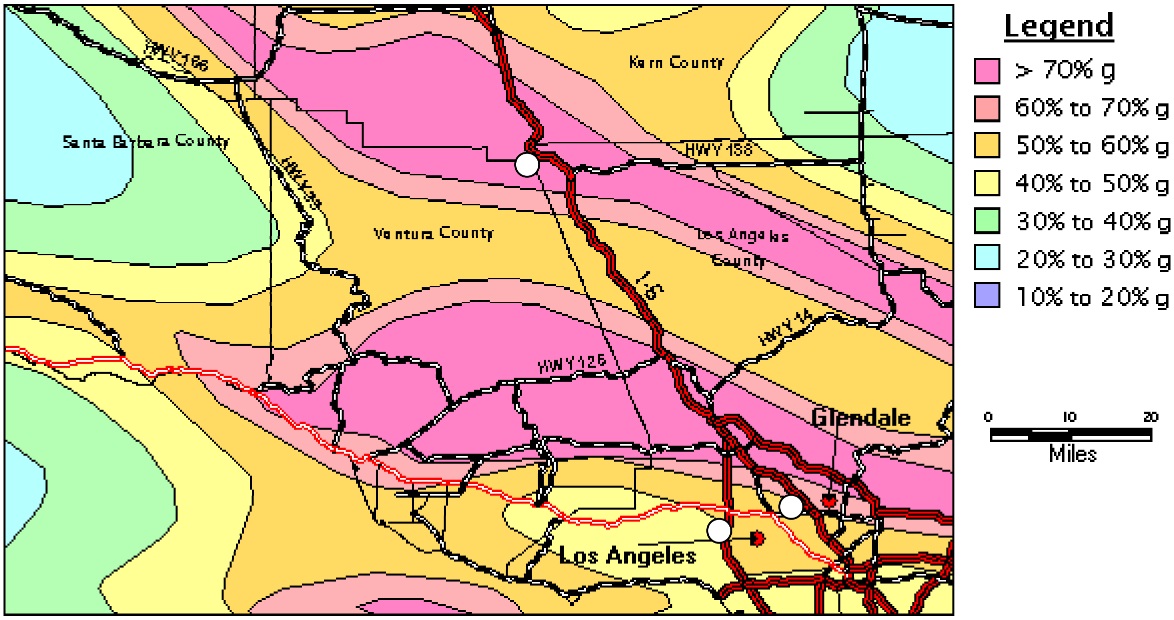

Fig. 5. Probabilistic Seismic Hazard Map by Petersen et al. (1996) for MHA on Soft Rock Site Condition at the 475-Year Hazard Level.

Calculations of seismic coefficient k for each of those sites are illustrated in Table 1. The values of MHA_r, $\overline{M}$, and $\bar{r}$ in Table 1 are obtained from the published maps indicated in the reference column. The f_{eq} values for the limiting displacement of 5 cm are seen to vary from 0.46 to 0.55, which are of the same order as the value used by Hynes-Griffin and Franklin (1984) for dams (f_{eq} = 0.5). The similarity of these f_{eq} values results from the compensating effects of the present procedure having smaller threshold displacements (which increases f_{eq}) and being formulated less conservatively (which decreases f_{eq}). Our values for u = 5 cm are smaller than the value used by Bray et al. (1998) for landfills (f_{eq} = 0.75) because our less conservative formulation overcompensates for our slightly smaller threshold displacements. The f_{eq} values for u = 15 cm are considerably smaller than those recommended by either Hynes-Griffin and Franklin or Bray et al. As discussed in the attached report, the majority of the Committee recommends the use of u = 5 cm for hillside construction, whereas a minority recommends a limiting u = 15 cm.

Table 1. Evaluation of Seismic Coefficient for Example Sites

				u = 5 cm		u = 15 cm		
	MHA_r (g)	$\overline{M}$	$\bar{r}$	f_{eq}	k	f_{eq}	k	Reference
Los Angeles	0.54	6.4	2.0	0.46	0.25	0.33	0.18	CDMG, 1998a
Glendale	0.65	7.0	7.0	0.49	0.34	0.38	0.25	CDMG, 1998b
Hwy 138 and I-5*	0.70	7.5-8.0	< 10 km	0.55	0.39	0.44	0.31	Petersen et al., 1996

* values approximate since no detailed map of this area

It should also be noted that the $\overline{M}$ values indicated in Table 1 are consistent with the characteristic earthquake magnitudes for faults near the respective sites (as tabulated in Petersen et al., 1996). The similarity of those magnitudes is the principal reason that the Committee does not consider it necessary to perform supplemental deterministic analyses of scenario events (which would have a magnitude similar to the characteristic earthquake magnitude).

Post-Screen Analysis

For sites that fail the screen analysis, more detailed slope displacement calculations should be performed. Several alternative analysis procedures are recommended by the Committee. Those include simplified analysis of Newmark displacement using the procedures formulated by Makdisi and Seed (1978) or Bray and Rathje (1998), or formal Newmark analysis of sliding block displacements using appropriate integration techniques with applicable earthquake time histories. Those procedures are well documented in the literature, and are summarized in Chapter 11 of the attached report.

CONCLUSIONS

In this appendix, we have presented a screen analysis procedure for seismic slope stability that takes into account local variations in seismicity, as represented by the magnitude (M) and distance (r) that most significantly contribute to the ground motion hazard at a site. The screen procedure is based on a statistical relationship previously developed by Bray and Rathje (1998) between seismic slope displacement (u), peak amplitude of shaking in the slide mass (k_{max}), significant duration of shaking ($D_{5\text{-}95}$), and the ratio of slope resistance to peak demand (k_y/k_{max}). The screen is formulated to separate sites expected to undergo small to negligible slope deformation from sites where larger and more damaging slope movements are likely. Application of the screen is straightforward. Pseudo-static seismic coefficient k is calculated using Eq. 1, with the parameter f_{eq} in Eq. 1 evaluated using Figure 4 based on the site seismicity and the tolerable slope displacement.

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