



SPECIAL PUBLICATION 117

GUIDELINES FOR EVALUATING AND MITIGATING SEISMIC HAZARDS IN CALIFORNIA

2008



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Cover photograph is of a storm-induced landslide in Laguna Beach, California. Earthquakes can also trigger landslides.

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Copies of these Guidelines, California's Seismic Hazards Mapping Act, and other related information are available on the World Wide Web at:

<http://www.conservation.ca.gov/cgs/shzp/Pages/Index.aspx>

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CHAPTER 1

INTRODUCTION

Prompted by damaging earthquakes in northern and southern California, in 1990 the State Legislature passed the Seismic Hazards Mapping Act. The Governor signed the Act, codified in the Public Resources Code as Division 2, Chapter 7.8 (see Appendix A), which became operative on April 1, 1991.

The purpose of the Act is to protect public safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure, and other hazards caused by earthquakes. The program and actions mandated by the Seismic Hazards Mapping Act closely resemble those of the Alquist-Priolo Earthquake Fault Zoning Act (which addresses only surface fault-rupture hazards) and are outlined below:

1. **The State Geologist** is required to delineate the various "seismic hazard zones."
2. **Cities and Counties**, or other local permitting authority, must regulate certain development "projects" within the zones. They must withhold the development permits for a site within a zone until the geologic and soil conditions of the project site are investigated and appropriate mitigation measures, if any, are incorporated into development plans.
3. **The State Mining and Geology Board** provides additional regulations, policies, and criteria, to guide cities and counties in their implementation of the law (see Appendix B). The Board also provides guidelines for preparation of the Seismic Hazard Zone Maps (available at <http://www.consrv.ca.gov/CGS/shezp/zoneguid.html>) and for evaluating and mitigating seismic hazards (this document).
4. **Sellers (and their agents)** of real property within a mapped hazard zone must disclose that the property lies within such a zone at the time of sale.

This document constitutes the guidelines for evaluating seismic hazards other than surface fault-rupture, and for recommending mitigation measures as required by Public Resources Code Section 2695(a). Nothing in these Guidelines is intended to conflict with or supersede any requirement, definition, or other provision of Chapter 7.8 of the Public Resources Code; California Code of Regulations, Title 14, Division 2, Chapter 8, Article 10; the Business and Professions Code; or any other state law or regulation.

Objectives

The objectives of these Guidelines are twofold:

1. To assist in the evaluation and mitigation of earthquake-related hazards for projects within designated zones of required investigations; and
2. To promote uniform and effective statewide implementation of the evaluation and mitigation elements of the Seismic Hazards Mapping Act.

The Guidelines will be helpful to the owner/developer seeking approval of specific development projects within zones of required investigation and to the engineering geologist and/or civil engineer who must investigate the site and recommend mitigation of identified hazards. They will also be helpful to the lead agency engineering geologist and/or civil engineer who must complete the technical review, and other lead agency officials involved in the planning and development approval process. Effective evaluation and mitigation ultimately depends on the combined professional judgment and expertise of the evaluating and reviewing professionals.

The methods, procedures, and references contained herein are those that the State Mining and Geology Board, the Seismic Hazards Mapping Act Advisory Committee, and its Working Groups believe are currently representative of quality practice. Seismic hazard assessment and mitigation is a rapidly evolving field and it is recognized that additional approaches and methods will be developed. If other methods are used, they should be justified with appropriate data and documentation.

For a general description of the Department's Seismic Hazards Zonation Program, its products and their uses, refer to the CGS website:

<http://www.conservation.ca.gov/cgs/shzp/Pages/Index.aspx>

CHAPTER 2

DEFINITIONS, CAVEATS, AND GENERAL CONSIDERATIONS

Definitions

Key terms that will be used throughout the Guidelines are defined in the Act and related regulations. These are:

- **"Acceptable level"** of risk means that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project [CCR Title 14, Section 3721(a)].
- **"Certified Engineering Geologist"** means an engineering geologist who is certified in the State of California [CCR Title 14, Section 3721(c); Business and Professions Code (BPC) Sections 7804 and 7822] and practicing in his or her area of expertise. These professionals will be referred to throughout these Guidelines as "engineering geologists." See page 8 (*Engineers or Geologists— Who Does What?*) for a discussion of scope of involvement in site-investigation reports and related reviews.
- **"Lead agency"** means the state agency, city, or county with the authority to approve projects [CCR Title 14, Section 3721(b)].
- **"Mitigation"** means those measures that are consistent with established practice and reduce seismic risk to "acceptable levels" [Public Resources Code (PRC) Section 2693(c)].
- **"Owner/Developer"** is defined as the party seeking permits to undertake a "project", as defined below.
- **"Project"** is defined by the Seismic Hazards Mapping Act as any structures for human occupancy, or any subdivision of land that contemplates the eventual construction of structures for human occupancy. Unless lead agencies impose more stringent requirements, single-family frame dwellings are exempt unless part of a development of four or more dwellings. (The definition is complex; see Table 1 for specific language.)
- **"Registered Civil Engineer"** means a civil engineer who is registered in the State of California [CCR Title 14, Section 3721(c); BPC Sections 6701-6704] and practicing in his or her area of expertise. These professionals will be referred to throughout these Guidelines as

"civil engineers." See page 8 (*Engineers or Geologists—Who Does What?*) for a discussion of scope of involvement in site-investigation reports and related reviews.

- **"Seismic Hazard Evaluation Reports"** document the data and methods used by the State Geologist to develop the **"Seismic Hazard Zone Maps."**
- **"Seismic Hazards Mapping Act"**— California Public Resources Code Sections 2690 and following, included as Appendix A.
- **"Seismic Hazards Mapping Regulations"**— California Code of Regulations (CCR), Title 14, Division 2, Chapter 8, Article 10, included as Appendix B.
- **"Seismic Hazard Zone Maps"** are maps issued by the State Geologist under PRC Section 2696 that show zones of required investigation.
- **"Site-Investigation Report"** means a report prepared by a certified engineering geologist and/or a civil engineer practicing within the area of his or her competence, which documents the results of an investigation of the site for seismic hazards and recommends mitigation measures to reduce the risk of identified seismic hazards to acceptable levels. In PRC Section 2693(b) and elsewhere, this report is referred to as a "geotechnical report."
- **"Zones of Required Investigation"** referred to as **"Seismic Hazard Zones"** in CCR Section 3722, are areas shown on Seismic Hazard Zone Maps where site investigations are required to determine the need for mitigation of potential liquefaction and/or earthquake-induced landslide ground displacements.

Definitions of technical terms appear in Appendix C.

Minimum Statewide Safety Standard

Based on the above definitions of "mitigation" and "acceptable risk," the Seismic Hazards Mapping Act and related regulations establish a statewide minimum public safety standard for mitigation of earthquake hazards. This means that the minimum level of mitigation for a project should reduce the risk of ground failure during an earthquake to a level that does not cause the collapse of buildings for human occupancy, but in most cases, **not** to a level of no ground failure at all. However, nothing in the Act, the regulations, or these Guidelines precludes lead agencies from enacting more stringent requirements, requiring a higher level of performance, or applying these requirements to developments other than those that meet the Act's definition of "project."

Areal Extent of Hazard

The Seismic Hazard Zone Maps are developed using a combination of historic records, field observations, and computer-mapping technology. The maps may not identify all areas that have potential for liquefaction, earthquake-induced landsliding, strong ground shaking, and other earthquake and geologic hazards. Although past earthquakes have caused ground failures in only

a small percentage of the total area zoned, a worst-case scenario of a major earthquake during or shortly after a period of heavy rainfall is something that has not occurred in northern California

TABLE 1. Definition of "Project"

Public Resources Code Section 2693.

As used in [Chapter 7.8, the Seismic Hazards Mapping Act]:

- d) "Project" has the same meaning as in Chapter 7.5 (commencing with Section 2621), except as follows:
 - (1) A single-family dwelling otherwise qualifying as a project may be exempted by the city or county having jurisdiction of the project.
 - (2) "Project" does not include alterations or additions to any structure within a seismic hazard zone which do not exceed either 50 percent of the value of the structure or 50 percent of the existing floor area of the structure.

Public Resources Code Section 2621.6.

- (a) As used in (Chapter 7.5, the Alquist-Priolo Earthquake Fault Zoning Hazard Act), "project" means either of the following:
 - (1) Any subdivision of land which is subject to the Subdivision Map Act (Division 2 (commencing with Section 66410) of Title 7 of the Government Code), and which contemplates the eventual construction of structures for human occupancy.
 - (2) Structures for human occupancy, with the exception of either of the following:
 - (A) Single-family wood-frame or steel-frame dwellings to be built on parcels of land for which geologic reports have been approved pursuant to paragraph (1).
 - (B) A single-family wood-frame or steel-frame dwelling not exceeding two stories when that dwelling is not part of a development of four or more dwellings.
- (b) For the purposes of this chapter, a mobile home whose body width exceeds eight feet shall be considered to be a single-family wood-frame dwelling not exceeding two stories.

California Code of Regulations Section 3601 (Policies and Criteria of the State Mining and Geology Board, With Reference to the Alquist-Priolo Earthquake Fault Zoning Act).

The following definitions as used within the Act and herein shall apply:

- (e) A "structure for human occupancy" is any structure used or intended for supporting or sheltering any use of occupancy, which is expected to have a human occupancy rate of more than 2,000 person-hours per year.
- (f) "Story" is that portion of a building included between the upper surface of any floor and the upper surface of the floor next above, except that the topmost story shall be that portion of the building included between the upper surface of the topmost floor and the ceiling or roof above. For the purpose of the Act and this subchapter, the number of stories in a building is equal to the number of distinct floor levels, provided that any levels that differ from each other by less than two feet shall be considered as one distinct level."

since 1906, and has not been witnessed in historic times in southern California. The damage from such an event in a heavily populated area is likely to be more widespread than that experienced in the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake, or the 1994 Northridge earthquake.

Off-Site Origin of Hazard

The fact that a site lies outside a zone of required investigation does not necessarily mean that the site is free from seismic or other geologic hazards, regardless of the information shown on the Seismic Hazard Zone Maps. The zones do not always include landslide or lateral spread run-out areas. Project sites that are outside of any zone may be affected by ground failure runoff from adjacent or nearby sites.

Finally, neither the information on the Seismic Hazard Zone Maps, nor in any technical reports that describe how the maps were prepared nor what data were used is sufficient to serve as a substitute for the required site-investigation reports called for in the Act.

Relationship of these Guidelines to Local General Plans and Permitting Ordinances

Public Resources Code Section 2699 directs cities and counties to "take into account the information provided in available seismic hazard maps" when it adopts or revises the safety element of the general plan and any land-use planning or permitting ordinances. Cities and counties should consider the information presented in these guidelines when adopting or revising these plans and ordinances.

Relationship of these Guidelines to the CEQA Process and Other Site Investigation Requirements

Nothing in these guidelines is intended to negate, supersede, or duplicate any requirements of the California Environmental Quality Act (CEQA) or other state laws and regulations. At the discretion of the lead agency, some or all of the investigations required by the Seismic Hazards Mapping Act may occur either before, concurrent with, or after the CEQA process or other processes that require site investigations.

Some of the potential mitigation measures described herein (e.g., strengthening of foundations) will have little or no adverse impact on the environment. However, other mitigation measures (e.g., draining of subsurface water, driving of piles, densification, extensive grading, or removal of liquefiable material) may have significant impacts. If the CEQA process is completed prior to the site-specific investigation, it may be desirable to discuss a broad range of potential mitigation measures (any that might be proposed as part of the project) and related impacts. If, however, part or all of the site-specific investigation is conducted prior to completion of the CEQA process, it may be possible to narrow the discussion of mitigation alternatives to only those that would provide reasonable protection of the public safety given site-specific conditions.

More stringent requirements are prescribed by the California Building Code (CCR Title 24) for hospitals, public schools, and essential service buildings. For such structures, the requirements of the Seismic Hazards Mapping Act are intended to complement the CCR Title 24 requirements.

Criteria for Project Approval

The State's minimum criteria required for project approval within zones of required investigation are defined in CCR Title 14, Section 3724, from which the following has been excerpted:

"The following specific criteria for project approval shall apply within seismic hazard zones and shall be used by affected lead agencies in complying with the provisions of the Act:

- (a) A project shall be approved only when the nature and severity of the seismic hazards at the site have been evaluated in a geotechnical report and appropriate mitigation measures have been proposed.*
- (b) The geotechnical report shall be prepared by a registered civil engineer or certified engineering geologist, having competence in the field of seismic hazard evaluation and mitigation. The geotechnical report shall contain site-specific evaluations of the seismic hazard affecting the project, and shall identify portions of the project site containing seismic hazards. The report shall also identify any known off-site seismic hazards that could adversely affect the site in the event of an earthquake. The contents of the geotechnical report shall include, but shall not be limited to, the following:*
 - (1) Project description.*
 - (2) A description of the geologic and geotechnical conditions at the site, including an appropriate site location map.*
 - (3) Evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice.*
 - (4) Recommendations for appropriate mitigation measures as required in Section 3724(a), above.*
 - (5) Name of report preparer(s), and signature(s) of a certified engineering geologist and/or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.*
- (c) Prior to approving the project, the lead agency shall independently review the geotechnical report to determine the adequacy of the hazard evaluation and proposed mitigation measures and to determine the requirements of Section 3724(a), above, are satisfied. Such reviews shall be conducted by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation."*

Lead agencies can have other, more stringent criteria for project approval. The State Mining and Geology Board recommends that the official professional Registration or Certification Number and license expiration date of each report preparer be included in the signature block of the

report. In addition, Chapter 3 provides a list of topics that should be addressed in site-investigation reports prepared for liquefaction and/or earthquake-induced landslides.

Engineers or Geologists - Who Does What?

The Act and Regulations state that the site-investigation reports 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 Hazards 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 Geophysicist Act, Section 7832; and Professional Engineers Act, Section 6704) restricts the practice of these two professions. Because of the differing expertise and abilities of engineering geologists and civil engineers, the scope of the site-investigation report for the project may require that **both** types of professionals prepare and review the report, each practicing in the area of his or her expertise.* Involvement of both engineering geologists and civil engineers will generally provide greater assurance that the hazards are properly identified, assessed, and mitigated.

The State Mining and Geology Board recommends that engineering geologists and civil engineers conduct the assessment of the surface and subsurface geological/geotechnical conditions at the site, including off-site conditions, to identify potential hazards to the project. It is appropriate for the civil engineer to design and recommend mitigation measures. It also is appropriate for both engineering geologists and civil engineers to be involved in the implementation of the mitigation measures— engineering geologists to confirm the geological conditions and civil engineers to oversee the implementation of the approved mitigation measures.

CHAPTER 3

OVERVIEW OF INVESTIGATIONS FOR ASSESSING SEISMIC HAZARDS

Introduction

Investigation of potential seismic hazards at a site can be performed in two steps or stages: (1) a preliminary **screening investigation**, and (2) a **quantitative evaluation** of the seismic hazard potential and its consequences. As noted below, it is possible to successfully complete the investigation by skipping one or the other stage. For example, a consultant's screening investigation may find that a previous site-specific investigation, on or adjacent to the project site, has shown that no seismic hazards exist, and that a quantitative evaluation is not necessary. Conversely, a consultant may know from experience that a project site is susceptible to a given hazard, and may opt to forego the screening investigation and start with a quantitative evaluation of the hazard.

Some lead agency reviewers recommend that for large projects the developer's consultant(s) meet with the lead agency technical reviewer prior to the start of the site investigation. This allows the consultant and technical reviewer to discuss the scope of the investigation. Topics of this discussion may include the area to be investigated for various hazards, the acceptability of investigative techniques to be used, on-site inspection requirements, or other local requirements.

Items to Consider in the Site Investigation Study

The following concepts are provided to help focus the site-investigation report:

1. Consultants are encouraged to utilize, if possible, the latest California Department of Conservation, Geological Survey (CGS) seismic ground-motion parameter data. This information is available at the following URLs: <http://www.consrv.ca.gov/cgs/rghm/pshamap/pshamain.html> or <http://eqint.cr.usgs.gov/deaggint/2002>.
2. The hazard zone mapping procedure for liquefaction and earthquake-induced landsliding utilizes state-of-the-art probabilistic ground-motion parameters developed jointly by the CGS and the U.S. Geological Survey, and published by the CGS (Cao and others, 2003; Frankel and others, 2002; Petersen and others, 1996).

2. The fact that a site lies within a mapped zone of required investigation does not necessarily indicate that a hazard requiring mitigation is present. Instead, it indicates that regional (that is, not site-specific) information suggests that the probability of a hazard requiring mitigation is great enough to warrant a site-specific investigation. However, the working premise for the planning and execution of a site investigation within Seismic Hazard Zones is that *the suitability of the site should be demonstrated*. This premise will persist until either: (a) the site investigation satisfactorily demonstrates the absence of liquefaction or landslide hazard, or (b) the site investigation satisfactorily defines the liquefaction or landslide hazard and provides a suitable recommendation for its mitigation.
3. The fact that a site lies outside a mapped zone of required investigation does not necessarily mean that the site is free from seismic or other geologic hazards, nor does it preclude lead agencies from adopting regulations or procedures that require site-specific soil and/or geologic investigations and mitigation of seismic or other geologic hazards. It is possible that development proposals may involve alterations (for example, cuts, fills, and/or modifications that would significantly raise the water table) that could cause a site outside the zone to become susceptible to earthquake-induced ground failure.
4. Lead agencies have the right to approve (and the obligation to reject) a proposed project based on the findings contained in the site-investigation report and the lead agency's technical review. The task of the developer's consulting engineering geologist and/or civil engineer is to demonstrate, to the satisfaction of the lead agency's technical reviewer, that:
 - The site-specific investigation is sufficiently thorough;
 - The findings regarding identified hazards are valid; and,
 - The proposed mitigation measures achieve an acceptable level of risk, as defined by the lead agency and CCR Title 14, Section 3721(a).

Screening Investigation

The purpose of screening investigations for sites within zones of required investigation is to evaluate the severity of potential seismic hazards, or to screen out sites included in these zones that have a low potential for seismic hazards. If a screening investigation can **clearly** demonstrate the absence of seismic hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required. If the findings of the screening investigation cannot demonstrate the absence of seismic hazards, then the more-comprehensive quantitative evaluation needs to be conducted.

The documents reviewed should be both regional and, if information is available, site-specific in scope. The types of information reviewed during a screening investigation often includes topographic maps, geologic and soil engineering maps and reports, aerial photographs, water well logs, agricultural soil survey reports, and other published and unpublished references. The references used should focus on current journals, maps, reports, and methods. Seismic Hazard

Evaluation Reports, which summarize the findings and data on which CGS's Seismic Hazard Zone Maps are based, can provide much of the regional geologic and seismic information needed for a screening investigation. Aerial photographs can be useful to identify existing and potential landslide and/or liquefaction features (headwall scarps, debris chutes, fissures, grabens, sand boils, etc.) that suggest or preclude the existence of ground failure hazards that might affect the site. Several sets of stereoscopic aerial photographs that pre-date project site area development, and taken during different seasons of the year are particularly useful for identifying subtle geomorphic features. A field reconnaissance of the area is highly recommended to verify the information developed in the earlier steps to fill in information in questionable areas, and to observe the surface features and details that could not be determined from other data sources.

Quantitative Evaluation of Hazard Potential

Detailed Field Investigations – General Information Needs

Within the zone of required investigations, the objective of the detailed field investigation is to obtain sufficient information on which the engineering geologist and/or civil engineer can evaluate the nature and severity of the risk and develop a set of recommendations for mitigation. In the case of projects where the property is to be subdivided and sold to others undeveloped, the aim of the investigation is to determine which parcels contain buildable sites that meet the previously defined acceptable level of risk. The work should be based upon a detailed, accurate topographic base map prepared by a registered civil engineer or land surveyor. The map should be of suitable scale, and should cover the area to be developed as part of the project, as well as adjacent areas: which affect or may be affected by the project.

The detailed field investigation commonly involves the collection of subsurface information from trenches or borings, on or adjacent to the site. The subsurface exploration should extend to depths sufficient to expose geologic and subsurface water conditions that could affect slope stability or liquefaction potential. A sufficient quantity of subsurface information is needed to permit the engineering geologist and/or civil engineer to extrapolate with confidence the subsurface conditions that might affect the project, so that the seismic hazard can be properly evaluated, and an appropriate mitigation measure can be designed by the civil.

The preparation of engineering geologic maps and geologic cross sections is often an important step to developing an understanding of the significance and extent of potential seismic hazards. These maps and/or cross sections should extend far enough beyond the site to identify off-site hazards and features that might affect the site.

Content of Reports

The site investigation report should contain sufficient information to allow the lead agency's technical reviewer to satisfactorily evaluate the potential for seismic hazards and the proposed mitigation. No attempt is made here to define the limits of what constitutes a complete screening investigation or quantitative evaluation report. Site-specific conditions and circumstances, as well as lead agency requirements, will dictate which issues and what level of detail are required

to adequately define and mitigate the hazard(s). The following list (Table 2) is provided to assist investigators and reviewers in identifying seismic hazard-related factors significant to the project. Not all of the information in the list will be relevant or required, and some investigations may require additional types of data or analyses.

Table 2. Recommended content for site-investigation reports within zones of required investigations.	
Reports that address liquefaction and/or earthquake-induced landslides should include, but not necessarily be limited to, the following data:	
1.	Description of the proposed project's location, topographic relief, drainage, geologic and soil materials, and any proposed grading.
2.	Site plan map of project site showing the locations of all explorations, including test pits, borings, penetration test locations, and soil or rock samples.
3.	Description of seismic setting, historic seismicity, nearest pertinent strong-motion records, and methods used to estimate (or source of) earthquake ground-motion parameters used in liquefaction and landslide analyses.
4.	1:24,000 or larger-scale geologic map showing bedrock, alluvium, colluvium, soil material, faults, shears, joint systems, lithologic contacts, seeps or springs, soil or bedrock slumps, and other pertinent geologic and soil features existing on and adjacent to the project site.
5.	Logs of borings, test pits, or other subsurface data obtained.
6.	Geologic cross sections depicting the most critical (least stable) slopes, geologic structure, stratigraphy, and subsurface water conditions, supported by boring and/or trench logs at appropriate locations.
7.	Laboratory test results; soil classification, shear strength, and other pertinent geotechnical data.
8.	Specific recommendations for mitigation alternatives necessary to reduce known and/or anticipated geologic/seismic hazards to an acceptable level of risk.
Reports that address earthquake-induced landslides may also need to include:	
1.	Description of shear test procedures (ASTM or other) and test specimens.
2.	Shear strength plots, including identification of samples tested, whether data points reflect peak or residual values, and moisture conditions at time of testing.
3.	Summary table or text describing methods of analysis, shear strength values, assumed groundwater conditions, and other pertinent assumptions used in the stability calculations.
4.	Explanation of choice of seismic coefficient and/or design strong-motion record used in slope stability analysis, including site and/or topographic amplification estimates.
5.	Slope stability analyses of critical (least-stable) cross sections, which substantiate conclusions and recommendations concerning stability of natural and as-graded slopes.
6.	Factors of safety against slope failure and/or calculated displacements for the various anticipated slope configurations (cut, fill, and/or natural slopes).
7.	Conclusions regarding the stability of slopes with respect to earthquake-induced landslides and their likely impact on the proposed project.
8.	Discussion of proposed mitigation measures, if any, necessary to reduce damage from potential earthquake-initiated landsliding to an acceptable level of risk.
9.	Acceptance testing criteria (e.g., pseudo-static factor of safety), if any, that will be used to demonstrate satisfactory remediation.
Reports that address liquefaction hazards may also need to include the following:	
1.	If methods other than Standard Penetration Test (SPT; ASTM D 1586; ASTM D 6066) and Cone Penetration Test (CPT; ASTM 3441) are used, description of pertinent equipment and procedural details of field measurements of penetration resistance (borehole type, hammer type and drop mechanism, sampler type and dimensions, etc.).
2.	Boring logs showing raw (unmodified) N-values if SPT's are performed; CPT probe logs showing raw qc-values and plots of raw sleeve friction if CPT's are performed.
3.	Explanation of the basis and methods used to convert raw SPT, CPT, and/or other non-standard data to "corrected" and "standardized" values.

**Table 2. Recommended content for site-investigation reports
within zones of required investigations.**

4. Tabulation and/or plots of corrected values used for analyses.
5. Explanation of methods used to develop estimates of field loading equivalent uniform cyclic stress ratios (CSR) used to represent the anticipated field earthquake excitation (cyclic loading).
6. Explanation of the basis for evaluation of the equivalent uniform cyclic stress ratio necessary to cause liquefaction (CRR) within the number of equivalent uniform loading cycles considered representative of the design earthquake.
7. Factors of safety against liquefaction at various depths and/or within various potentially liquefiable soil units.
8. Conclusions regarding the potential for liquefaction and its likely impact on the proposed project.
9. Discussion of proposed mitigation measures, if any, necessary to reduce potential damage caused by liquefaction to an acceptable level of risk.
10. Criteria for SPT-based, CPT-based, or other types of acceptance testing, if any, that will be used to demonstrate satisfactory remediation.

CHAPTER 4

ESTIMATION OF EARTHQUAKE GROUND-MOTION PARAMETERS

Introduction

Quantitative analyses of in-situ liquefaction resistance and earthquake-induced landslide potential requires site-specific assessment of ground shaking levels suitable for those purposes. A simplified Seed-Idriss (1982) liquefaction analysis requires an estimation of peak ground acceleration (PGA) and earthquake magnitude. A pseudo-static slope stability analysis may require estimates of PGA and magnitude for the selection of an appropriate seismic coefficient. If a seismic site response analysis is needed, a Newmark analysis or a more complex dynamic analysis is to be performed, representative strong-motion records will need to be selected on the basis of site-specific ground-motion parameter estimates. The following sections of this Chapter provide guidance on the selection of site-specific ground-motion parameters and representative strong-motion records.

Simple Prescribed Parameter Values (SPPV)

Probabilistic ground-motion parameter values on firm rock for PGA, predominant magnitude, and distance in the form of statewide maps have been jointly prepared by CGS and the U.S. Geological Survey for a 10 percent probability of exceedance over a 50-year period (Cao and others, 2003; Frankel and others, 2002). Since these maps are updated every few years, the most current versions are made available online, with bedrock PGA and predominant magnitude and distance available from the USGS at the following URL: <http://eqint.cr.usgs.gov/deaggint/2002>, and PGA with soil factors applied from CGS at the following URL: <http://www.consrv.ca.gov/cgs/rghm/pshamap/pshamain.html>.

Alternatively, a site response analysis can be performed (e.g. using a SHAKE program or other more sophisticated site response code). Bedrock PGA and predominant magnitude and distance obtained online can be used to select an appropriate strong-motion record for input into the site response analysis.

PGA estimated by the above procedures may still require additional adjustment to account for topographic and basin effects. Use of the SPPV method is not recommended for sites located very near to seismic sources, where reliable ground-motion estimates may require consideration of near-field source effects.

Probabilistic Seismic Hazard Analysis (PSHA)

Site-specific probabilistic seismic hazard analyses can be performed, and can supersede the SPPV-values of PGA for seismic hazard studies, even if PSHA studies result in adoption of a lower level of seismic ground motion. PSHA studies typically include the following:

1. A database consisting of potentially damaging earthquake sources, including known active faults and historic seismic source zones, their activity rates, and distances from the project site. This should include a comparison with CGS-developed slip rates for faults considered in the CGS statewide probabilistic seismic hazard map. Differences in slip rates should be documented and the reasons for them explained (for example, revised slip rates or new paleoseismic information from recent studies). CGS recommends that their earthquake source database be used directly, because it is updated regularly and is readily available online: <http://www.consrv.ca.gov/cgs/rghm/psha/index.htm>;
2. Use of published maximum moment magnitudes for earthquake sources, or estimates that are justified, well-documented, and based on published procedures;
3. Use of published curves (or those used by CGS) for attenuation of PGA with distance from earthquake source, as a function of earthquake magnitude and travel path (e.g., see special issue of Earthquake Spectra, v. 24, n.1, 2008);
4. An evaluation of the likely effects of site-specific response characteristics (e.g., amplification due to soft soils, deep sedimentary basins, topography, near-source effects, etc.); and,
5. Characterization of the ground motion at the site in terms of PGA with a 10 percent probability of exceedance in 50 years, taking into account historical seismicity, available paleoseismic data, the geological slip rate of regional active faults, and site-specific response characteristics.

Useful references include Reiter, 1990; National Research Council, 1988; Hayes, 1985; Algermissen and others, 1982; Cornell, 1968; Youngs and Coppersmith, 1985; Working Group on California Earthquake Probabilities, 1990, 1995, and 2008; Okumura and Shinozuka, 1990; and Kramer, 1996.

Deterministic Seismic Hazard Analysis (DSHA)

Deterministic evaluation of seismic hazard can also be performed, and the results of correctly performed and suitably comprehensive DSHA studies can also supersede SPPV values of PGA. DSHA studies typically include the following:

1. Evaluation of potentially damaging earthquake sources, and deterministic selection of one or more suitable "controlling" sources and seismic events. The deterministic earthquake event magnitude for any fault should be a *maximum* value that is specific to that seismic source. Maximum earthquakes may be assessed by estimating rupture dimensions of the fault (e.g.,

Wells and Coppersmith, 1994; dePolo and Slemmons, 1990). The CGS database of earthquake sources is readily available (see section on PSHA);

2. Use of published curves for the effects of seismic travel path using the shortest distance from the source(s) to the site (e.g., see special issue of Earthquake Spectra, v. 24, n.1, 2008); and,
3. Evaluation of the effects of site-specific response characteristics on either (a) site accelerations, or (b) cyclic shear stresses within the site soils of interest.

Selection of a Site-Specific Design Strong-Motion Record

Depending on the method used to perform a seismic slope stability or liquefaction analysis, it may be necessary to select design strong-motion records that represent the anticipated earthquake shaking at a project site. For a seismic slope-stability analysis the design strong-motion record will be used to evaluate the site seismic response (site amplification) and/or for the calculation of Newmark displacements. For liquefaction evaluations the design strong-motion record will be used for the site seismic response to determine the appropriate peak ground acceleration to use in a simplified Seed-Idriss-type liquefaction analysis. It could also be used for a detailed finite-element strain-based method of analysis where the magnitude of potential large lateral spread displacements are critical to the proposed project.

The selection process typically involves two steps: (1) estimating magnitude, source distance, spectral character and peak ground acceleration parameters relevant to the project site, and (2) searching for existing strong-motion records that have parameters that closely match the estimated values. The methods described in the preceding sections of this chapter describe the recommended approaches to the parameter estimates. The selection of a representative strong-motion record should consider the following:

1. The selection should be based primarily on matching recordings for equivalent magnitude, source distance, site conditions (including soil type, topography, and potential for basin resonance and directivity effects), and PGA, that have spectral characteristics relevant to the project;
2. It may not always be possible to find a good match between the site parameters and the existing strong-motion records, and it may be necessary to use a record that does not match the site parameter criteria and scale it to fit those parameters, making sure that the duration of the scaled record is appropriate for the anticipated magnitude;
3. If the site to be analyzed is underlain by soils or weakly cemented rock, and a strong-motion recording site with similar characteristics cannot be found, a seismic site response analysis should be performed as mentioned above; and,
4. It is recommended that several strong-motion records be used to account for natural variability of earthquake ground motions. The selection of strong-motion records requires consideration of the controlling earthquake magnitude, distance, site conditions, and other effects such as forward-directivity. Due to the important influence of the characteristics of

strong-motion records on the results of dynamic analyses, a suite of at least five records is suggested for most projects.

A database of strong-motion records is available from the National Earthquake Engineering Center at the following URL: <http://www.quake.ca.gov/cisn-edc/> and the Pacific Earthquake Engineering Research Center Strong Ground Motion Database used to derive the next-generation attenuation relations: http://peer.berkeley.edu/products/strong_ground_motion_db.html. This and other sources for acquiring strong-motion records are provided in Appendix D.

CHAPTER 5

ANALYSIS OF EARTHQUAKE-INDUCED LANDSLIDE HAZARDS

Screening Investigations for Earthquake-Induced Landslide Potential

The purpose of screening investigations for sites within zones of required investigation for earthquake-induced landslides is to evaluate the severity of the hazard, or to screen out sites included in these zones that have a low potential for landslide hazards. If a screening investigation can *clearly* demonstrate the absence of earthquake-induced landslide hazard at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement and no further investigation will be required. If the findings of the screening investigation cannot demonstrate the absence of the hazard, then the more-comprehensive quantitative evaluation needs to be conducted.

An important aspect of evaluating the potential for earthquake-induced landslides is the recognition of the types of slope failures commonly caused by earthquakes. Keefer (1984) studied 40 historical earthquakes and found that different types of landslides occur with different frequencies. Table 3 summarizes Keefer's findings. In addition, Keefer (1984) summarized the geologic environments that are likely to produce earthquake-induced landslides. A table of these environments is provided in Appendix E to assist in the evaluation of project sites for the screening investigations.

The screening investigation should evaluate, and the report should address, the following basic questions:

- **Are existing landslides, active or inactive, present on, or adjacent (either uphill or downhill) to the project site?**

An assessment of the presence of existing landslides on the project site for a screening investigation will typically include a review of published and unpublished geologic and landslide inventory maps of the area and an interpretation of aerial photographs. The distinctive landforms associated with landslides (scarps, troughs, disrupted drainages, etc.) should be noted, if present, and the possibility that they are related to landslides should be assessed.

Relative abundance of earthquake-induced landslides from 40 historical earthquakes (Keefer, 1984; Table 4, p. 409).	
Relative Abundance of Earthquake-Induced Landslides	Description
Very Abundant (more than 100,000 in the 40 earthquakes)	Rock falls, disrupted soil slides, rock slides
Abundant (10,000 to 100,000 in the 40 earthquakes)	Soil lateral spreads, soil slumps, soil block slides, soil avalanches
Moderately common (1000 to 10,000 in the 40 earthquakes)	Soil falls, rapid soil flows, rock slumps
Uncommon (100 to 1000 in the 40 earthquakes)	Subaqueous landslides, slow earth flows, rock block slides, rock avalanches

- **Are there geologic formations or other earth materials located on or adjacent to the site that are known to be susceptible to landslides?**

Many geologic formations in California, notably late Tertiary and Quaternary siltstones and shales (for example, the Orinda and Modelo formations), are highly susceptible to landsliding. These rock units are generally well known among local engineering geologists. For some areas, susceptible formations have also been noted on the Landslide Hazard Identification Maps published by CGS.

- **Do slope areas show surface manifestations of the presence of subsurface water (springs and seeps), or can potential pathways or sources of concentrated water infiltration be identified on or upslope of the site?**

Subsurface water in slopes can be an important indicator of landslide potential. Water may be forced to the surface along impermeable layers such as landslide rupture surfaces. Springs, seeps, or vegetation (phreatophytes) may result from impermeable layers and near-surface water. Topographic depressions, heavy irrigation, or disrupted surface water channels can cause ponding and increased infiltration of surface water. These features may be visible on pre- and/or post-development aerial photographs taken during certain seasons, or during a field reconnaissance. Presence of shallow subsurface water is significant because pore-water pressure reduces the forces resisting landslide movement.

- **Are susceptible landforms and vulnerable locations present? These include steep slopes, colluvium-filled swales, cliffs or banks being undercut by stream or wave action, areas that have recently slid.**

In addition to existing landslide deposits, certain other slopes are especially susceptible to landsliding. These include very steep slopes, and ones where the support at the base of the slope has been removed or reduced. Removal of support at the base of a slope occurs naturally by stream or wave erosion and the same effect can be produced by grading of cut slopes. Colluvium-filled swales usually develop naturally over thousands of years, and the resulting thick, deeply weathered soil may be especially susceptible to debris flows. Hazardous slope

features can generally be noted on aerial photographs, sufficiently detailed topographic maps, or from a geologic field reconnaissance.

- **Given the proposed development, could anticipated changes in the surface and subsurface hydrology (due to watering of lawns, on-site sewage disposal, concentrated runoff from impervious surfaces, etc.) increase the potential for future landsliding in some areas?**

Misdirected runoff from streets during rainstorms can cause saturation of surficial materials and, in turn, development of catastrophic debris flows. Improperly designed highway culverts and watering of lawns on marine terraces can create unstable gullies, undermined coastal bluffs, or both. It is likely that the proposed development will alter the local groundwater regime in some way. The investigation should describe the likely effects that altered runoff patterns, lawn watering or septic systems will have on slope stability; identify sensitive areas; and, when warranted, recommend mitigation.

Additional Considerations

The Earthquake-Induced Landslides Working Group recommends that the screening investigation should include a site reconnaissance by the project's engineering geologist and/or civil engineer. This will allow for the recognition of potential earthquake hazards that cannot normally be recognized in a purely office-based screening investigation.

The next section includes quantitative screening procedures that serve to identify when slide displacements are likely to be significant enough to warrant a full quantitative analysis of seismic slope stability.

Guidance on the preparation of a report for the screening investigation is provided in Chapter 3 of these Guidelines. If the results of the screening investigation show that the potential for earthquake-induced landsliding is low, the report should state the reasons why a quantitative evaluation is not needed for the project site.

Quantitative Evaluation of Earthquake-Induced Landslide Potential

If the screening investigation indicates the presence of potentially unstable slopes affecting the proposed project site, a quantitative evaluation of earthquake-induced landslide potential should be conducted. The major phases of such a study typically includes a detailed field investigation, drilling and sampling, geotechnical laboratory testing, and slope stability analyses. Reference should be made to Chapter 3 for guidance on what types of information from the following sections should be included in the site-investigation report.

Detailed Field Investigation

Engineering Geologic Investigations

The engineering geologic investigation phase of the project site investigation consists of surface observations and geologic mapping. The overall scope of the engineering geologic investigation for earthquake-induced landslide hazards is fundamentally the same as the work that would be conducted for any project that has potential landslide hazards, regardless of the triggering mechanism. However, the investigator should keep in mind the environments and the relative abundance of landslide types triggered by earthquakes as described by Keefer (1984) and shown in Appendix E and Table 3, respectively. The engineering geologic investigation is significant because it provides the basis for the subsurface investigations, field instrumentation, and geotechnical analyses that follow.

Prior to the site reconnaissance, the area of the project and possible details about the proposed construction (structures and cut/fill locations) should be identified, and available geologic and geotechnical information, stereoscopic aerial photographs, and topographic maps should be collected and reviewed (Keaton and DeGraff, 1996). If a screening investigation has been conducted for the site, much of this information may already have been reviewed. Once the results of the office-based investigation have been completed and understood, on-site engineering geologic mapping can be conducted.

The purpose of the on-site engineering geologic mapping is to document surface conditions, which, in turn, provides a basis for projecting subsurface conditions that may be relevant to the stability of the site. The on-site engineering geologic mapping should identify, classify, and locate on a map the features and characteristics of existing landslides, and surficial and bedrock geologic materials, especially those landslides and geologic materials that may specifically impact the proposed construction activities. Other important aspects of the site to document include: landslide features and estimates of depth to the rupture surface; distribution and thickness of colluvium; rock discontinuities such as bedding, jointing, fracturing and faulting; depth of bedrock weathering; surface water features such as streams, lakes, springs, seeps, marshes, and closed or nearly closed topographic depressions.

Engineering geologic cross sections should be located so as to provide information that will be needed for planning subsurface investigations and stability analyses. The most useful orientation is typically perpendicular to topographic contours and longitudinally down existing landslide deposits. The projected shape of the rupture surface, geologic contacts and orientations, and groundwater surfaces should be shown along with the topographic profile. Estimates of the depth to the landslide rupture surface is an important parameter for planning a subsurface investigation and longitudinal cross sections can be helpful in making these estimates (McGuffey and others, 1996).

The results of the engineering geologic mapping can be presented in many forms, but generally should include a map, cross sections, and proposed construction and subsurface investigation locations and/or field instrumentation sites. Whatever method of presentation is chosen, it should be remembered that the presentation of the surface mapping information needs to be

characterized in terms that are meaningful for, and usable by the design engineer. Doing so will help ensure that key factors that must be accommodated in the construction are understood (Keaton and DeGraff, 1996).

Subsurface Investigation

Planning

Exploratory work by the engineering geologist and civil engineer should be conducted at locations considered most likely to reveal any subsurface conditions which may indicate the potential for earthquake-induced landslide failures, especially those that directly impact the proposed project. In particular, an investigation should locate and define the geometry of bedding and fracture surfaces, contacts, faults, and other discontinuities as well as actual landslide rupture surfaces.

Subsurface exploration methods can be classed as direct methods and indirect methods (Hunt, 1984a). Direct methods, such as test borings and the excavation of test pits or trenches, allow the examination of the earth materials, usually with the removal of samples. Indirect methods, such as geophysical surveys and the use of the cone penetrometer, provide a measure of material properties that allows the estimation of the material type (McGuffey and others, 1996).

Subsurface investigations should be supervised by an experienced engineering geologist and/or civil engineer to ensure that the field activities are properly executed and the desired results are achieved. According to McGuffey and others (1996), the subsurface investigation field supervision should:

1. Ensure that technical and legal contract specifications are followed;
2. Maintain liaison with the designer of the exploration program;
3. Select and approve modifications to the program as new or unanticipated conditions are revealed;
4. Ensure that complete and reliable field reports are developed; and
5. Identify geologic conditions accurately.

The depth to which explorations should extend can be difficult to define in advance of the subsurface investigation. Cross sections from a surface engineering geological investigation can be helpful in planning the depths of excavations required in a subsurface investigation. In general, borings or other direct investigative techniques should extend deep enough (a) to identify materials that have not been subjected to movements in the past but might be involved in future movements, and (b) to clearly identify underlying stable materials. The exploration program plan should be flexible enough to allow for expanding the depth of investigation when the data obtained suggest deeper movements are possible (McGuffey and others, 1996).

Samples and Sampling

Soil and rock samples are taken primarily for laboratory strength and compressibility tests and for the measurement of in-situ material properties. Samples that may be obtained from subsurface borings and excavations belong to one of two basic categories: disturbed and undisturbed samples. Disturbed samples are collected primarily for soil classification tests where the preservation of the soil structure is not essential, or for remolding in the laboratory and subsequent strength and compressibility tests. Undisturbed samples do not entirely represent truly undisturbed soil or rock conditions because the process of sampling and transporting inevitably introduces some disturbance into the soil or rock structure.

Samples of the weathered and/or colluvial soil, the existing landslide rupture materials, and the weakest components of rock units should be taken for laboratory measurement of engineering properties. Special care should be taken to obtain oriented samples of existing zones of weakness or rupture surfaces. For shallow landslides it may be possible to expose and sample critical zones of weakness in the walls of trenches or test pits. For deep-seated landslides it often is extremely difficult to sample the zones of weakness with small-diameter, geotechnical drilling equipment, and it may be appropriate to consider using bucket auger drilling and down-hole geologic logging and sampling techniques (Scullin, 1994).

It is the responsibility of the field supervising geologist or engineer to accurately label and locate the collected samples. He or she is also responsible for the proper transportation of collected samples, particularly undisturbed samples, to prevent sample disturbance by excessive shaking, allowing samples to dry or slake, or by exposing samples to heat or freezing conditions. A large variety of soil boring techniques and sampler types is available. A detailed explanation of the many types is beyond the scope of these Guidelines, but is readily available in the literature (Hvorslev, 1948; ASTM, 2008; U.S. Bureau of Reclamation, 1974 and 1989; U.S. Navy, 1986; Hunt, 1984a; Krynine and Judd, 1957; Acker, 1974; Scullin, 1994; Johnson and DeGraff, 1988; McGuffey and others, 1996; Blake and others, 2002).

Subsurface Water

The presence of subsurface water can be a major contributing factor to the dynamic instability of slopes and existing landslides. Therefore, the identification and measurement of subsurface water in areas of suspected or known slope instability should be an integral part of the subsurface investigation. The location and extent of groundwater, perched groundwater and potential water barriers should be defined and identified in cross-sectional view. Subsurface water conditions within many landslides are best considered as complex, multiple, partially connected flow systems. McGuffey and others (1996) have listed the following recommendations:

1. Surface observations are essential in determining the effect of subsurface water on landslide instability.
2. Periodic or seasonal influx of surface water to subsurface water will not be detected unless subsurface water observations are conducted over extended time periods.

3. Landslide movements may open cracks and develop depressions at the head of a landslide that increase the rate of infiltration of surface water into the slide mass.
4. Ponding of surface water anywhere on the landslide may cause increased infiltration of water into the landslide and should be investigated.
5. Disruption of surface water channels and culverts may also result in increased infiltration of surface water into the landslide.
6. Landslide movements may result in blockage of permeable zones that were previously freely draining. Such blockage may cause a local rise in the groundwater table and increased saturation and instability of the landslide materials. Subsurface observations should therefore be directed to establishing subsurface water conditions in the undisturbed areas surrounding the landslide.
7. Low permeability soils, which are commonly involved in landslides, have slow response times to changes in subsurface water conditions and pressures. Long-term subsurface water monitoring is required in these soils.
8. Accurate detection of subsurface water in rock formations is often difficult because shale or claystone layers, intermittent fractures, and fracture infilling may occlude subsurface water detection by boring or excavation.
9. Borings should never be the only method of subsurface water investigation; nevertheless they are a critical component of the overall investigation.

Geotechnical Laboratory Testing

The geotechnical testing of soil and rock materials typically follows accepted published standards (ASTM, 2008; Head, 1989). Good professional judgment is expected in the selection of appropriate samples, shear tests, and interpretation of the results in arriving at strength characteristics appropriate for the present and anticipated future slope conditions. The following guidelines are provided for evaluating soil properties:

1. Soil properties, including unit weight and shear strength parameters (cohesion and friction angle), may be based on appropriate conventional laboratory and field tests.
2. Testing of earth materials should be in accordance with the appropriate ASTM Standards that are updated annually (ASTM, 2008).
3. Prior to shear tests, samples should be soaked a sufficient length of time to approximate a saturated moisture condition.
4. Stability analyses generally should use the lowest values derived from the suite of samples tested.

5. Residual test values should be used for static analysis of existing landslides, along shale bedding planes, highly distorted bedrock, over-consolidated fissured clays, and for paleosols and topsoil zones under fill. Peak values may be used for pseudo-static or dynamic calculations if the buildup of pore pressures is not anticipated and if permitted by the lead agency. Consideration of reducing the strength values for dynamic analyses should be made in light of the measured material properties and anticipated subsurface water conditions (see section on Effective-Stress vs. Total-Stress Conditions below).
6. Appropriate analyses of existing failures (back-calculated strengths) in slopes similar to that under consideration in terms of height, geology, and soil or rock materials may be helpful in determining or verifying proposed shear strength parameters.
7. Laboratory shear strength values used for design of fill slopes steeper than two horizontal to one vertical (2:1) and for buttress fills should be verified by testing during slope grading. In the event that the shear strength values from field samples are less than those used in design, the slope should be reanalyzed and modified as necessary to provide the required factor of safety for stability.

Effective-Stress vs. Total-Stress Conditions

In principle, a pseudo-static or Newmark analysis can be performed on either a total-stress or effective-stress basis. The geotechnical industry practice for ‘typical’ developments has been to determine shear strength parameters from direct shear tests and perform the analysis assuming drained soil conditions, where the effective stresses are known and static and dynamic shear strengths are considered the same. For most investigations where the slopes are unsaturated or partially saturated, this assumption will be valid and the results of the analysis will tend to be conservative. However, for saturated slopes this assumption ignores the build-up of pore pressures due to dynamic loading, which can lower the shear resistance to failure and, in some cases, result in unconservative stability evaluations. For such conditions a total-stress analysis is required that assumes undrained soil response.

Seed (1966) presented an approach to a total-stress analysis for earth embankments that uses dynamic shear tests to derive a factor of safety that accounts for (a) initial conditions; (b) changes in stress and reorientation of principal stress; (c) decrease in strength due to cyclic loading conditions; and (d) decrease in strength due to undrained conditions during earthquake loading. This method is rigorous, and provides good estimates of the dynamic behavior of saturated materials but may be too costly for most projects.

A simpler approach to a total stress analysis would be to determine total-stress strength parameters (i.e., $c = S_u$, $\phi = 0$ for saturated conditions; values of c , ϕ for partially saturated conditions) from undrained triaxial shear tests and use those values in the stability analysis. Jibson and Keefer (1993) showed how to conduct such an analysis, and their results indicated that factors of safety and critical slip surfaces differed significantly from those generated from an effective-stress analysis. The U.S. Army Corps of Engineers practice is to use a composite shear strength envelope (based on a consolidated-drained test at low confining pressures and a

consolidated-undrained test at high confining pressures) for permeable soils, and a consolidated-undrained strength envelope for soils with low permeability (Hynes and Franklin, 1984).

Slope Stability Analysis

General Considerations

Slope stability analysis will generally be required by the lead agency for cut, fill, and natural slopes whose slope gradient is steeper than two horizontal to one vertical (2:1), and on other slopes that possess unusual geologic conditions such as unsupported discontinuities or evidence of prior landslide activity. Analysis generally includes deep-seated and surficial stability evaluation under both static and dynamic (earthquake) loading conditions.

Evaluation of deep-seated slope stability should be guided by the following:

1. The potential failure surface used in the analysis may be composed of circles, planes, wedges or other shapes considered to yield the minimum factor of safety against sliding for the appropriate soil or rock conditions. The potential failure surface having the lowest factor of safety should be sought.
2. The anticipated placement of fill material based on the proposed grading plan should be the incorporated. The areas of grading should be the focus area for the slope stability analysis.
3. Forces to be considered include the gravity loads of the potential failure mass, structural surcharge loads and supported slopes, and loads due to anticipated earthquake forces. The potential for hydraulic head (or significant pore-water pressure) should be evaluated and its effects included when appropriate. Total unit weights for the appropriate soil moisture conditions are to be used.

Evaluation of surficial slope stability should be guided by the following:

1. Structural discontinuities in the steep bedrock slopes should be evaluated for rock fall and rockslide type failures. Comparing these potential zones of weakness to the slope geometry can be done using stereo plot diagrams and available rock-failure software.
2. Calculations may be based either on analysis procedures for stability of an infinite slope with seepage parallel to the slope surface or on another method acceptable to the lead agency. For the infinite slope analysis, the minimum assumed depth of soil saturation is the smaller of either a depth of one (1) meter or depth to firm bedrock. Soil strength characteristics used in analysis should be obtained from representative samples of surficial soils that are tested under conditions approximating saturation and at normal loads approximating conditions at very shallow depth.

3. Ravines, swales, and hollows on natural slopes warrant special attention as potential sources of fast-moving debris flows and other types of landslides. If possible, structures should be located away from the base or axis of these types of features.

In both deep-seated and surficial slope stability evaluations, appropriate mitigation procedures and stabilization should be recommended, in order to provide the required level of slope stability. Recommendations for mitigation of damage to the proposed development caused by failure of off-site slopes should be made unless slope-specific investigations and analyses demonstrate that the slopes are stable.

Analysis Methods

The first and simplest approach to a dynamic slope stability calculation is a **pseudo-static analysis**, in which the earthquake load is simulated by an "equivalent" static horizontal acceleration acting on the mass of the landslide, in a limit-equilibrium analysis (Terzaghi, 1950; Janbu, 1973; Bromhead, 1986; Nash, 1987; Chowdhury, 1978; Morgenstern and Sangrey, 1978; Seed, 1979; Hunt, 1984b; Duncan, 1996). The pseudo-static approach has certain limitations (Cotecchia, 1987; Kramer, 1996), but this methodology is considered to be generally conservative, and is the one most often used in current practice. Expanding the work of Bray and Rathje (1998), Blake and others (2002) and Stewart and others (2003) have developed a procedure to select a pseudo-static seismic coefficient on the basis of site-specific ground motion parameters that can serve as an initial "screening analysis" to determine whether a more rigorous analysis is warranted. Using an expanded worldwide ground motion database, this approach has been refined into an integrated and statistically robust method of selecting seismic coefficients based on spectral acceleration and a specified displacement threshold (Bray and Travararou, 2007).

The second procedure is known as a **Newmark or cumulative displacement analysis** (Newmark, 1965; Makdisi and Seed, 1978; Wilson and Keefer, 1983; Hynes and Franklin, 1984; Jibson, 1993; Bray, 2007). The procedure involves the calculation of the "yield" acceleration, which is the horizontal acceleration that causes the slide mass' static factor of safety to reach 1.0, usually from a limit-equilibrium slope stability analysis. The procedure then uses earthquake strong-motion records, modified if necessary for site conditions or scaled to match seismological conditions, which are numerically integrated twice to calculate the cumulative displacement of the sliding mass relative to the ground below the slip surface. These analytical displacements are then evaluated in light of the slope material properties and the requirements of the proposed development. The pseudo-static and Newmark displacement analyses will be described in more detail in the following sections.

Pseudo-Static Analysis

The ground –motion parameter used in a pseudo-static analysis is referred to as the seismic coefficient "k." The selection of a seismic coefficient has relied heavily on engineering judgment and local code requirements because there is no simple method for determining an appropriate value.

Cautionary Note: *The seismic coefficient "k" is **not** equivalent to the peak horizontal ground acceleration value, either probabilistic or deterministic; therefore PGA should not be used as a seismic coefficient in pseudo-static analyses. The use of PGA will usually result in overly conservative factors of safety (Seed, 1979; Chowdhury, 1978). Furthermore, the practice of reducing the PGA by a "repeatable acceleration" factor to obtain a pseudo-static coefficient has no basis in the scientific or engineering literature.*

There have been a number of published articles that provide guidance in the selection of an appropriate seismic coefficient for pseudo-static analyses. Seed's 1979 article (the 19th Rankine Lecture) summarizes the factors to be considered in evaluating dynamic stability of earth-and rock-fill embankments. After evaluating all of the available data on earthquake-induced deformations of embankment dams, Seed recommended some basic guidelines for making preliminary evaluations of embankments to ensure acceptable performance (i.e., permanent deformations which would not imperil the overall structural integrity of an embankment dam). These recommendations were: using a pseudo-static coefficient of 0.10 for magnitude 6½ earthquakes and 0.15 for magnitude 8¼ earthquakes, with an acceptable factor of safety of the order of 1.15. Seed believed that his guidelines would ensure that permanent ground deformations would be acceptably small. Seed also made extensive commentary on the choice of appropriate material strengths, and limited his recommendations to those embankments composed of materials that do not undergo severe strength loss due to seismic shaking with an expected crest acceleration of less than 0.75g.

The limitations to selecting seismic coefficients on the basis of these references are twofold. First, the magnitude of acceptable displacements for earth embankments, roughly one meter, is far greater than what is acceptable for structures meant for human occupancy. Second, they only peripherally account for differences in earthquake magnitude and distance at differing sites, implying that resulting stability analyses will be over-conservative in some cases and under-conservative in others.

To address these significant limitations, Blake and others (2002) and Stewart and others (2003) used the simplified design procedures developed by Bray and others (1998) to develop a "screen analysis procedure," based on a pseudo-static approach, that accounts for the anticipated seismicity at a site and allows for different levels of acceptable displacements. By their formulation, the seismic coefficient, k_{eq} , is derived from,

$$k_{eq} = f_{eq} * (MHA_r / g)$$

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

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

where NRF is a factor that accounts for the nonlinear response of the materials above the slide plane; u is displacement; and D_{5-95} is the duration of strong shaking, a function of earthquake magnitude and distance.

Blake and others (2002) have simplified the process of estimating f_{eq} for ranges of magnitude and distance by preparing sets of curves for two displacement (u) values, 5 cm and 15 cm. These curves are reproduced in Figure 1.

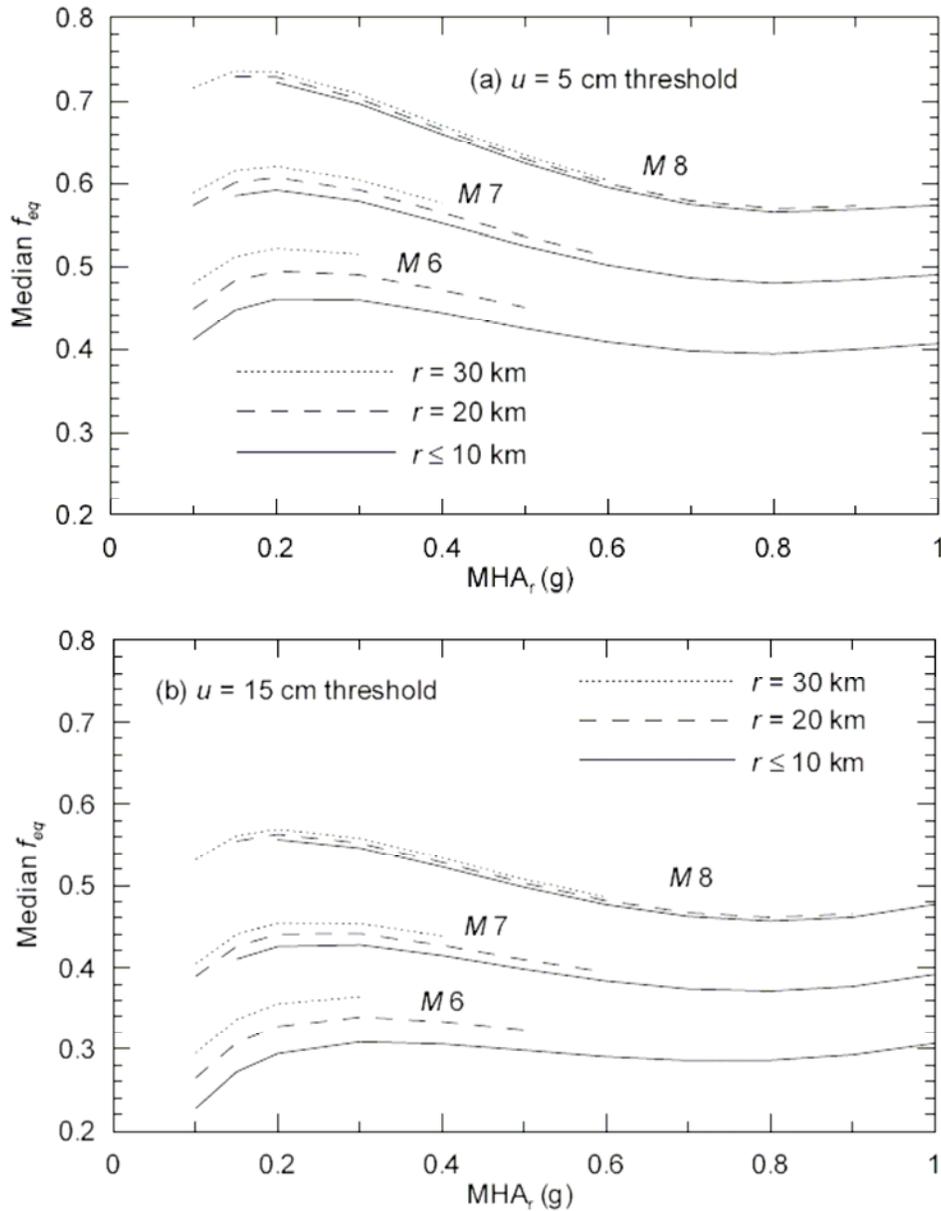


Figure 1. Values of f_{eq} as a Function of MHA_r , Magnitude and Distance for Threshold Displacements of (a) 5 cm and (b) 15 cm (Modified from Blake and others, 2002).

The value of threshold displacement used to determine f_{eq} should be based on consideration of the quality of the site geotechnical information used, the level of understanding of the underlying geologic materials and their discontinuities, the significance of the project development (critical facilities), and the value considered acceptable by the local regulatory agency. Blake and others (2002) provide a thorough description of how their screening analysis procedure is performed, and Stewart and others (2003) provide the rationale for the procedure.

Newmark Displacement Analysis

A Newmark displacement analysis consists of three basic steps, as outlined below:

1. The first step is to perform a limit-equilibrium stability analysis to determine the location and shape of the critical slip surface (the slip surface with the lowest factor of safety), and the yield acceleration (a_y), defined as the acceleration required to bring the factor of safety to 1.0. Most computer-based slope stability programs include iterative routines for finding both of these parameters. If a computer program with these options is not available, the critical slip surface can be obtained through iterative trial-and-error, and the yield acceleration can be calculated from Newmark's relation

$$a_y = (FS - 1)g \sin \alpha$$

where FS is the static factor of safety, g is the acceleration due to gravity, and α is the angle from the horizontal that the center of mass of the landslide first moves.

2. The second step is to select acceleration time histories that represent the expected ground motions at the project site. The selection process typically involves estimating magnitude, source-to-site distance, peak ground acceleration and spectral response relevant to the project site, and searching for existing strong-motion records that have parameters that closely match the estimated values. Methods for determining these site parameters and selecting a representative strong-motion record are outlined in Chapter 4. For Newmark analyses, Jibson (1993) recommended using: (1) Arias Intensity (Wilson and Keefer, 1985; Wilson, 1993; Travararou and others, 2003), (2) magnitude and source distance, and (3) PGA and duration as criteria for selecting a suite of strong-motion records having characteristics of interest at a project site. In general, site-specific PGA should be derived from a probabilistic seismic hazard analysis for a 475-year return period, and mode magnitude and mode distance derived from de-aggregation of that hazard level. Analysis of multiple records spanning a range of estimated shaking characteristics produces a range of calculated displacements, which provides a better basis for judgment of slope performance than one displacement calculated from a single record that may have unique idiosyncrasies. If the slopes to be analyzed are composed of soils or weakly cemented rock, and a strong-motion recording with similar site characteristics cannot be found, a seismic site response analysis should be performed. Bray and Rathje (1998) described a method of using a one-dimensional wave propagation program (e.g., SHAKE91, Idriss and Sun, 1992; SHAKE, Schnabel and others, 1972) to find the average response at the slip surface prior to calculating displacements as described in Chapter 4, sources for acquiring strong-motion records are provided in Appendix D.

3. The final step in a Newmark analysis is to calculate the cumulative displacements anticipated for the landslide under investigation. Computer software capable of calculating displacements from strong-motion records is available (Jibson and Jibson, 2003) and can greatly simplify the analysis.

Types of Sliding Block Models

Newmark (1965) regarded a landslide mass as a rigid body and showed that both infinite slope and circular failures could be modeled as a rigid block on an inclined plane. A rigid-block sliding model is appropriate for displacement analyses for relatively thin slide masses, such as what occurs on natural slopes during earthquakes. However, the rigid-block assumption can be very un-conservative for thicker, compliant slope-forming materials (Kramer and Smith, 1997; Rathje and Bray, 1999; Wartman, and others, 2003).

Makdisi and Seed (1978) accounted for the effects of earthquake ground shaking within a slide mass by first performing a dynamic response analysis for an earth embankment, and then double integrating the resulting time histories to estimate cumulative displacements at different locations in the embankment – a process referred to as a “decoupled analysis.” Modern computers and commercially available software have allowed for more frequent use of dynamic models that calculate displacements considering the effects of ground motion input from below the slide mass and the effects of ground motions within the slide mass – referred to as a “fully coupled analysis.” Wartman and others (2003) have defined the parameters where rigid-block, decoupled and fully coupled sliding block analyses are most appropriate.

Simplified Newmark Displacement Estimation Procedures

Several simplified procedures have been developed for estimating Newmark displacements, and in some cases have been found to produce reasonable results for actual landslides (Pradel and others, 2005). Jibson (2007) prepared regression models for Newmark rigid sliding block displacement in terms of yield acceleration ratio (a_y/PGA), yield acceleration ratio and earthquake magnitude, Arias Intensity and yield acceleration, and Arias Intensity and yield acceleration ratio. Bray and others (1998) and Rathje and Bray (1998) developed a simplified procedure based on a large number of strong motion records processed through a decoupled displacement analysis. This procedure requires as input rock PGA, input rock mean period, input rock significant duration (D_{5-95}), yield acceleration, slide mass thickness, and average shear wave velocity. Bray and Travararou (2007) developed a semi-empirical procedure by processing over 600 strong motion records through a one-dimensional fully coupled non-linear displacement analysis. This procedure requires estimates of the initial fundamental period of the sliding mass (T_s) and its yield coefficient (k_y), site-specific acceleration response spectral value at $1.5 T_s$ (S), and the controlling earthquake magnitude, (M), to estimate the likely seismic displacement, (D in cm), with a probability of exceedance of 50% or lower as:

$$\begin{aligned}
 - \quad \ln(D) &= 11.10 - 2.83 \ln(k_y) - 0.333(\ln(k_y))^2 + 0.566 \ln(k_y)\ln(S) \\
 - \quad &+ 3.04\ln(S) - 0.244(\ln(S))^2 + 1.5T_s + 0.278(M - 7) \\
 - \quad &
 \end{aligned}$$

Saygili and Rathje (2008) developed regression models for Newmark rigid sliding block displacement based on over 2,000 strong motion records. The models are a function of the yield acceleration and different combinations of ground motion parameters, including peak ground acceleration (PGA), peak ground velocity (PGV), Arias Intensity, and Mean Period.

Evaluation of Potential Earthquake-Induced Landslide Hazards

The determination of dynamic slope stability (i.e., pseudo-static factors of safety or analytical displacements), and the acceptable parameters used in the analysis, should follow the standards defined by the lead agency. If no standards exist, the following general values may be used for defining the stability of slopes for static and dynamic loads.

Pseudo-Static Analysis

Slopes that have a pseudo-static factor of safety greater than 1.0 using a seismic coefficient derived from the screening analysis procedure of Stewart and others (2003), can be considered stable. If the pseudo-static analysis results in a factor of safety lower than 1.0, the project engineer can either employ a Newmark displacement analysis (or other displacement-type analysis method if acceptable to the lead agency) to determine the magnitude of slope displacements, or design appropriate mitigation measures.

Newmark Displacement Analysis

The Newmark analysis models a highly idealized and simplistic failure mechanism; thus, as discussed previously, the calculated displacements should be considered order-of-magnitude estimates of actual field behavior. Rather than being an accurate guide of observable landslide displacement in the field, Newmark displacements provide an index of probable seismic slope performance, and considerable judgment is required in evaluating seismic stability in terms of Newmark displacements. In some jurisdictions, less than 10 cm is considered stable, whereas, more than 30 cm is considered unstable. As a general guideline,

1. Newmark displacements of 0 to 15 cm are unlikely to correspond to serious landslide movement and damage.
2. In the 15 to 100 cm range, slope deformation may be sufficient to cause serious ground cracking or enough strength loss to result in continuing (post-seismic) failure. Determining whether displacements in this range can be accommodated safely requires good professional judgment that takes into account issues such as landslide geometry and material properties.
3. Calculated displacements greater than 100 cm are very likely to correspond to damaging landslide movement, including possible catastrophic failure, and such slopes should be considered unstable.

CHAPTER 6

ANALYSIS OF LIQUEFACTION HAZARDS

Screening Investigations for Liquefaction Potential

The purpose of screening investigations for sites within zones of required investigation for liquefaction is to determine whether a given site has obvious indicators of a low potential for liquefaction failure (e.g., bedrock near the surface or deep ground water without perched water zones), or whether a more comprehensive field investigation is necessary to determine the potential for damaging ground displacements during earthquakes.

If a screening investigation can *clearly* demonstrate the absence of liquefaction hazards at a project site, and if the lead agency technical reviewer concurs with this finding, the screening investigation will satisfy the site-investigation report requirement. If there is a reasonable expectation that liquefiable soils exist on the site and the engineering geologist and/or civil engineer can demonstrate that large lateral spread displacements (of more than 0.5 meter) are unlikely (e.g., Youd and others, 2002; Bardet and others, 2002; Zhang and others, 2004), the local agency may give them the option to forego the quantitative evaluation of liquefaction hazards and provide a structural mitigation for certain classes of structures. These mitigation methods are outlined in the mitigation section of this chapter. If the findings of the investigation fall outside these two options, then the more-comprehensive quantitative evaluation described below needs to be conducted.

Screening investigations for liquefaction hazards should address the following basic questions:

- **Are potentially liquefiable soil types present?**

Given the highly variable nature of Holocene deposits that are likely to contain liquefiable materials, most sites will require borings to determine whether liquefiable materials underlie the project site. Borings used to define subsurface soil properties for other purposes (e.g., foundation investigations, environmental or groundwater studies) may provide valuable subsurface geologic and/or geotechnical information.

The vast majority of liquefaction hazards are associated with sandy soils and silty soils of low plasticity. Cohesive soils are generally not considered susceptible to soil liquefaction. Analysis of fine-grained soils affected by the 1999 earthquakes in Taiwan and Turkey has led to rejection of the so-called “Chinese Criteria”, particularly grain size, as an indicator of potential soil failure. Although soils having a plasticity index (PI) greater than 7 have generally been expected

to behave like clays (Boulanger and Idriss, 2006), Bray and Sancio (2006) found loose soils with a PI < 12 and moisture content > 85% of the liquid limit are susceptible to liquefaction. Moreover, sensitive soils having PI > 18 can undergo severe strength loss, so engineering judgment must be applied when using screening criteria. It is recommended that both PI and moisture content criteria be considered for screening purposes.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction. Most gravelly soils drain relatively well, but when: (a) their voids are filled with finer particles, or (b) they are surrounded by less pervious soils, drainage can be impeded and they may be vulnerable to cyclic pore pressure generation and liquefaction. Gravelly geologic units tend to be deposited in a more-turbulent depositional environment than sands or silts, tend to be fairly dense, and so generally resist liquefaction. Accordingly, conservative "preliminary" methods may often suffice for evaluation of their liquefaction potential. For example, gravelly deposits that can be shown to be pre-Holocene in age (older than about 11,000 years) are generally not considered susceptible to liquefaction.

- **If present, are the potentially liquefiable soils saturated or might they become saturated?**

In order to be susceptible to liquefaction, potentially liquefiable soils must be saturated or nearly saturated. In general, liquefaction hazards are most severe in the upper 50 feet of the surface, but on a slope near a free face or where deep foundations go beyond that depth, liquefaction potential should be considered at greater depths. If it can be demonstrated that any potentially liquefiable materials present at a site: (a) are currently unsaturated (e.g., are above the water table), (b) have not previously been saturated (e.g., are above the historic-high water table), and (c) are highly unlikely to become saturated (given foreseeable changes in the hydrologic regime), then such soils generally do not constitute a liquefaction hazard that would require mitigation. Note that project development, changes in local or regional water management patterns, or both, can significantly raise the water table or create zones of perched water. Extrapolating water table elevations from adjacent sites does not, by itself, demonstrate the absence of liquefaction hazards, except in those unusual cases where a combination of uniformity of local geology and very low regional water tables permits very conservative assessment of water table depths. Screening investigations should also address the possibility of local "perched" water tables, the raising of water levels by septic systems, or the presence of locally saturated soil units at a proposed project site.

- **Is the geometry of potentially liquefiable deposits such that they pose significant risks requiring further investigation, or might they be mitigated by relatively inexpensive foundation strengthening?**

Relatively thin seams of liquefiable soils (on the order of only a few centimeters thick), if laterally continuous over sufficient area, can represent potentially hazardous planes of weakness and sliding, and may thus pose a hazard with respect to lateral spreading and related ground displacements. Thus, the screening investigation should identify the presence of gently sloping ground and nearby free faces (cut slopes, stream banks, and shoreline areas), whether on or off-site, to determine whether lateral spreading and related ground displacements might pose a

hazard to the project. If such features are found, the quantitative evaluation of liquefaction usually will be warranted because of potential life-safety concerns.

Even when it is not possible to demonstrate the absence of potentially liquefiable soils or prove that such soils are not and will not become saturated, it may be possible to demonstrate that any potential liquefaction hazards can be adequately mitigated through a simple strengthening of the foundation of the structure, as described in the mitigation section of this chapter, or other appropriate methods.

- **Are in-situ soil densities sufficiently high to preclude liquefaction?**

If the screening evaluation indicates the presence of potentially liquefiable soils, either in a saturated condition or in a location that might subsequently become saturated, then the resistance of these soils to liquefaction and/or significant loss of strength due to cyclic pore pressure generation under seismic loading should be evaluated. If the screening investigation does not conclusively eliminate the possibility of liquefaction hazards at a proposed project site (a factor of safety of 1.5 or greater), then more extensive studies are necessary.

A number of investigative methods may be used to perform a screening evaluation of the resistance of soils to liquefaction. These methods are somewhat approximate, but in cases wherein liquefaction resistance is very high (e.g., when the soils in question are very dense) then these methods may, by themselves, suffice to adequately demonstrate sufficient level of liquefaction resistance, eliminating the need for further investigation. It is emphasized that the methods described in this section are more approximate than those discussed in the quantitative evaluation section, and so require very conservative application.

Methods that satisfy the requirements of a screening evaluation, at least in some situations, include:

1. Direct in-situ relative density measurements, such as the ASTM D 1586-92 and ASTM D6066-96e1 (Standard Penetration Test [SPT]) or ASTM D3441-94 (Cone Penetration Test [CPT]).
2. Preliminary analysis of hydrologic conditions (e.g., current, historical and potential future depth(s) to subsurface water). Current groundwater level data, including perched water tables, may be obtained from permanent wells, driller's logs and exploratory borings. Historical groundwater data can be found in reports by various government agencies, although such reports often provide information only on water from production zones and ignore shallower water.
3. Non-standard penetration test data. It should be noted that correlation of non-standard penetration test results (e.g., sampler size, hammer weight/drop, hollow stem auger) with SPT resistance is very approximate, and so requires very conservative interpretation, unless direct SPT and non-standard test comparisons are made at the site and in the materials of interest.

4. Geophysical measurements of shear-wave velocities.
5. "Threshold strain" techniques represent a conservative basis for screening of some soils and some sites (National Research Council, 1985). These methods provide only a very conservative bound for such screening, however, and so are conclusive only for sites where the potential for liquefaction hazards is very low.

Quantitative Evaluation of Liquefaction Resistance

Liquefaction investigations are best performed as part of a comprehensive investigation. These Guidelines are to promote uniform evaluation of the resistance of soil to liquefaction.

Detailed Field Investigation

Engineering Geologic Investigations

Engineering geologic investigations should determine:

1. The presence, texture (e.g., grain size), and distribution (including depth) of unconsolidated deposits;
2. The age of unconsolidated deposits, especially for Quaternary Period units (both Pleistocene and Holocene Epochs);
3. Zones of flooding or historic liquefaction; and,
4. The groundwater level to be used in the liquefaction analysis, based on data from well logs, boreholes, monitoring wells, geophysical investigations, or available maps. Generally, the historic high groundwater level should be used unless other information indicates a higher or lower level is appropriate.

The engineering geologic investigations should reflect relative age, soil classification, three-dimensional distribution and general nature of exposures of earth materials within the area. Surficial deposits should be described as to general characteristics (including environment of deposition) and their relationship to present topography and drainage. It may be necessary to extend the mapping into adjacent areas. Geologic cross sections should be constrained by boreholes and/or trenches when available.

Geotechnical Field Investigation

The vast majority of liquefaction hazards are associated with sandy and/or silty soils. For such soil types, there are at present two approaches available for quantitative evaluation of the soil's resistance to liquefaction. These are: (1) correlation and analyses based on in-situ Standard Penetration Test (SPT) (ASTM D1586-92 and ASTM D6066-96e1) data, and (2) correlation and analyses based on in-situ Cone Penetration Test (CPT) (ASTM D3441-94) data. Both of these

methods have some relative advantages (see Table 4). Either of these methods can suffice for some site conditions, but there is also considerable advantage to using them jointly.

Seed and others (1985) provide guidelines for performing "standardized" SPT, and also provide correlations for conversion of penetration resistance obtained using most of the common alternate combinations of equipment and procedures in order to develop equivalent "standardized" penetration resistance values — $(N_1)_{60}$. These "standardized" penetration resistance values can then be used as a basis for evaluating liquefaction resistance.

Table 4. Comparative advantages of SPT and CPT methods.	
SPT ADVANTAGES	CPT ADVANTAGES
Retrieves a sample. This permits identification of soil type <i>with certainty</i> , and permits evaluation of fines content (which influences liquefaction resistance). Note that CPT provides poor resolution with respect to soil classification, and so usually requires some complementary borings with samples to more reliably define soil types and stratigraphy.	Provides <i>continuous</i> penetration resistance data, as opposed to averaged data over discrete increments (as with SPT), and so is less likely to "miss" thin layers and seams of liquefiable material.
Liquefaction resistance correlation is based primarily on field case histories, and the vast majority of the field case history database is for in-situ SPT data	Faster and less expensive than SPT, as no borehole is required.

Cone penetration test (CPT) tip resistance (q_c) may also be used as a basis for evaluation of liquefaction resistance, by either (a) direct empirical comparison between q_c data and case histories of seismic performance (Olsen, 1988), or (b) conversion of q_c -values to "equivalent" $(N_1)_{60}$ -values and use of correlations between $(N_1)_{60}$ data and case histories of seismic performance. At present, Method (b) — conversion of q_c to equivalent $(N_1)_{60}$ — is preferred because the field case history database for SPT is well developed compared to CPT correlations. A number of suitable correlations between q_c and $(N_1)_{60}$ are available (e.g., Robertson and Campanella, 1985; Seed and De Alba, 1986). These types of conversion correlations depend to some extent on knowledge of soil characteristics (e.g., soil type, mean particle size (D_{50}), fines content). When the needed soil characteristics are either unknown or poorly defined, then it should be assumed that the ratio

$$\frac{q_c (kg / cm^2)}{N (blows / ft)}$$

is approximately equal to five for conversion from q_c to "equivalent" N-values.

The simplified procedure for evaluating liquefaction resistance is becoming more standardized (Youd and others, 2001), and is continuing to evolve as new data from post-earthquake investigations around the world improve correlations between liquefaction and penetration resistance (e.g. Cetin and others, 2004; Moss and others, 2006; Idriss and Boulanger, 2008). These developments should be followed and new, improved correlations used as appropriate.

Geotechnical Laboratory Testing

The use of laboratory testing (e.g., cyclic triaxial, cyclic simple shear, cyclic torsional tests) on "undisturbed" soil samples as the sole basis for the evaluation of in-situ liquefaction resistance is not recommended, as unavoidable sample disturbance and/or sample densification during reconsolidation prior to undrained cyclic shearing causes a largely unpredictable, and typically unconservative, bias to such test results. Laboratory testing is recommended for determining grain-size distribution (including mean grain size D_{50} , effective grain size D_{10} , and percent passing #200 sieve), unit weights, moisture contents, void ratios, and relative density.

In addition to sandy and silty soils, some gravelly soils are potentially vulnerable to liquefaction (Evans and Fragaszy, 1995, Evans and Zhou, 1995). Most gravelly soils drain relatively well, but when their voids are filled with finer particles, or they are surrounded (or "capped") by less pervious soils, drainage can be impeded and they may be vulnerable to liquefaction. Gravelly soils tend to be deposited in a more turbulent environment than sands or silts, and are fairly dense, and so are generally resistant to liquefaction. Accordingly, conservative "preliminary evaluation" methods (e.g., geologic assessments and/or shear-wave velocity measurements) often suffice for evaluation of their liquefaction potential. When preliminary evaluation does not suffice, more accurate quantitative methods must be used. Unfortunately, neither SPT nor CPT provides reliable penetration resistance data in soils with high gravel content, as the large particles impede these small-diameter penetrometers. At present, the best available technique for quantitative evaluation of the liquefaction resistance of coarse, gravelly soils involves correlations and analyses based on in-situ penetration resistance measurements using the very large-scale Becker-type Hammer system (Harder, 1988).

Evaluation of Potential Liquefaction Hazards

The factor of safety for liquefaction resistance has been defined:

$$\text{Factor of Safety} = \frac{CRR}{CSR}$$

where CSR is the cyclic stress ratio generated by the anticipated earthquake ground motions at the site, and CRR is the cyclic stress ratio required to generate liquefaction (Seed and Idriss, 1982). For the purposes of evaluating the results of a quantitative assessment of liquefaction potential at a site, a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk. This factor of safety assumes that high-quality,

site-specific penetration resistance and geotechnical laboratory data were collected, and that the most recent ground-motion data from CGS/USGS (<http://eqint.cr.usgs.gov/deaggint/2002> or <http://www.consrv.ca.gov/cgs/rghm/pshamap/pshamain.html>.) were used in the analyses. If lower factors of safety are calculated for some soil zones, then an evaluation of the level (or severity) of the hazard associated with potential liquefaction of these soils should be made.

Such hazard assessment requires considerable engineering judgment. The following is, therefore, only a guide. The assessment of hazard associated with potential liquefaction of soil deposits at a site must consider two basic types of hazard: 1) translational site instability (sliding, edge failure, lateral spreading, flow failure, etc.) that potentially may affect all or large portions of the site, and 2) more localized hazard at and immediately adjacent to the structures and/or facilities of concern (e.g., bearing failure, settlement, localized lateral movements).

As Bartlett and Youd (1995) have stated: "Two general questions must be answered when evaluating the liquefaction hazards for a given site:

- (1) 'Are the sediments susceptible to liquefaction?' and
- (2) 'If liquefaction does occur, what will be the ensuing amount of ground deformation?'"

Lateral Spreading and Site Displacement Hazards

Lateral spreading on gently sloping ground generally is the most pervasive and damaging type of liquefaction failure (Bartlett and Youd, 1995). Assessment of the potential for lateral spreading and other large site displacement hazards may involve the need to determine the residual undrained strengths of potentially liquefiable soils. If required, this should be done using in-situ SPT or CPT test data (e.g., Seed and Harder, 1990). The use of laboratory testing for this purpose is not recommended, as a number of factors (e.g., sample disturbance, sample densification during reconsolidation prior to undrained shearing, and void ratio redistribution) render laboratory testing a potentially unreliable, and, therefore, unconservative basis for assessment of in-situ residual undrained strengths. Assessment of residual strengths of silty or clayey soils may, however, be based on laboratory testing of "undisturbed" samples.

Assessment of potential lateral spread hazards must consider dynamic loading as a potential "driving" force, in addition to gravitational forces. It should again be noted, that relatively thin seams of liquefiable material, if fairly continuous over large lateral areas, might serve as significant planes of weakness for translational movements. If prevention of translation or lateral spreading is ascribed to structures providing "edge containment," then the ability of these structures (e.g., berms, dikes, sea walls) to resist failure must also be assessed. Special care should be taken in assessing the containment capabilities of structures prone to potentially "brittle" modes of failure (e.g., brittle walls which may break, tiebacks which may fail in tension). If a hazard associated with potentially large translational movements is found to exist, then either: (a) suitable recommendations for mitigation of this hazard should be developed, or (b) the proposed "project" should be discontinued.

When suitably sound lateral containment is demonstrated to prevent potential sliding on liquefied layers, then potentially liquefiable zones of finite thickness occurring at depth may be deemed to pose no significant risk beyond the previously defined minimum acceptable level of risk. Suitable criteria upon which to base such an assessment include those proposed by Ishihara (1985, Figure 88; 1996, Chapter 16).

For information on empirical models that might be appropriate to use in these analyses, see Youd and others (2002), Bardet and others (2002), and Idriss and Boulanger (2008).

Localized Liquefaction Hazards

If it can be shown that no significant risk of large translational movements exists, or if suitable mitigation measures can be developed that address such risks, then studies should proceed to consideration of five general types of more localized potential hazards, including:

1. ***Potential foundation bearing failure, or large foundation settlements due to ground softening and near-failure in bearing.*** To form a basis for concluding that no hazard exists, a high factor of safety ($FS > 1.5$) should be based on a realistic appraisal of the minimum soil strengths likely to be mobilized to resist bearing failure (including residual undrained strengths of soils considered likely to liquefy or to suffer significant strength loss due to cyclic pore pressure generation). If such hazard does exist, then appropriate recommendations for mitigation of this hazard should be developed.
2. ***Potential structural and/or site settlements.*** Settlements for saturated and unsaturated clean sands can be estimated using simplified empirical procedures (e.g., Tokimatsu and Seed, 1987; Ishihara and Yoshimine, 1992). These procedures, developed for relatively clean, sandy soils, have been found to provide reasonably reliable settlement estimates for sites not prone to significant lateral spreading. Improved relations suitable for spreadsheet analysis are available that are based on fines-corrected penetration resistance (Idriss and Boulanger, 2008).

Any prediction of liquefaction-related settlements is necessarily approximate, and related hazard assessment and/or development of recommendations for mitigation of such hazard should, accordingly, be performed with suitable conservatism. Similarly, it is very difficult to reliably estimate the amount of localized differential settlement likely to occur as part of the overall predicted settlement: localized ***differential*** settlements on the order of up to two-thirds of the total settlements anticipated should be assumed unless more precise predictions of differential settlements can be made.

3. ***Localized lateral displacement; "lateral spreading" and/or lateral compression.*** Methods for prediction of lateral ground displacements due to liquefaction-related ground softening are not yet well supported by data from case histories of field performance. As such case history data are now being developed, significant advances in the reliability and utility of techniques for prediction of lateral displacements may be expected over the next few years. Finite element models represent the most sophisticated method currently in use for

calculating permanent displacements due to liquefaction lateral spreading. Like the dynamic analysis for landslide displacements, this method evaluates time histories of the stresses and strains for a strong-motion time history. This method is a state-of-the-art approach to liquefaction hazards and will likely take time to become the state-of-the-practice.

Consultants performing liquefaction hazard assessment should do their best to keep abreast of such developments (e.g. Idriss and Boulanger, 2008). At present, lateral ground displacement magnitudes can be predicted with reasonable accuracy and reliability only for cases wherein such displacements are likely to be "small" (e.g., on the order of 15 cm or less). Larger displacements may be predicted with an accuracy of \pm one meter or more; this level of accuracy may suffice for design of some structures (e.g., earth and rock-fill dams), but does not represent a sufficiently refined level of accuracy as to be of use for design of foundations for most types of structures.

It may be possible to demonstrate that localized lateral displacements will be 0.5 meter or less based on: (a) evaluation of soil stratigraphy, residual undrained strengths, and duration and severity of seismic loading, or (b) simplified empirical methods. Youd and others (2002) and Bardet and others (2002) empirical procedures use an existing field case history database of lateral spread occurrences. Other empirical methods, such as those based on estimating permanent shear strains in liquefied zones, or more complex analyses, may yield somewhat different results but should be allowed if the methods are documented and the results justified. When likely maximum lateral displacements can be shown to be less than 0.5 meter it may be possible to design foundations with sufficient strength to withstand the expected movements without complete failure. In all other cases, more extensive recommendations are needed for mitigation of the hazard associated with potential lateral displacements.

4. ***Floatation of light structures with basements, or underground storage structures.*** Light structures, which extend below the groundwater table and contain large void spaces, may "float" or rise out of the ground during, or soon after an earthquake. Structures that are designed for shallow groundwater conditions typically rely on elements, such as cantilevered walls or tie-downs, that resist the buoyant or uplift forces produced by the water. If the material surrounding these elements liquefies, the resisting forces can be significantly reduced and the entire structure may be lifted out of the ground.
5. ***Hazards to Lifelines.*** To date, most liquefaction hazard investigations have focused on assessing the risks to commercial buildings, homes, and other occupied structures. However, liquefaction also poses problems for streets and lifelines—problems that may, in turn, jeopardize lives and property. For example, liquefaction locally caused natural gas pipelines to break and catch fire during the Northridge earthquake, and liquefaction-caused water line breakage greatly hampered firefighters in San Francisco following the 1906 earthquake. Thus, although lifelines are not explicitly mentioned in the Seismic Hazards Mapping Act, cities and counties may wish to require investigation and mitigation of potential liquefaction-caused damage to lifelines.

CHAPTER 7

GUIDELINES FOR MITIGATING SEISMIC HAZARDS

Introduction

Michael Scullin wrote the consummate book on grading practices in southern California (Scullin, 1983). His book describes the prevailing excavation, grading, inspection and code enforcement techniques used to construct engineered fill in Los Angeles and Orange County during the mid-1970's. Beginning in 1990, the Seismic Hazards Mapping Act (SHMA) began requiring mitigation for landslide and liquefaction hazard found in zones of required investigation. This chapter presents specific mitigation measures for earthquake-induced landslides and liquefaction summarized from the previous version of Special Publication 117, information from the two publications listed below, and various techniques described in professional workshops conducted over the past 10 years (Seed, 1998):

1. "Recommended Procedures for Implementation of CGS Special Publication 117- Guidelines for Analyzing and Mitigating Liquefaction Hazards in California", (Martin and Lew, 1999); and,
2. "Recommended Procedures for Implementation of CGS Special Publication 117-Guidelines for Analyzing and Mitigating Landslide Hazards in California" (Blake and others, 2002).

Although there are numerous structural and foundation treatments available to decrease earthquake damages, the focus of this chapter is on the use of grading techniques, surface and ground water control and geotechnical ground improvement methods to reduce the hazards of earthquake- induced ground failures. There are projects that are clearly not feasible because of safety limits or economic constraints where the grading needed to ensure stability is either very complex or involves excessive yardages. However, there is a whole range of stability conditions that can be feasibly mitigated. Earthquake induced hazards can be treated in three general ways:

1. **Avoid the Hazard:** Where the potential for failure is beyond an acceptable level of safety during the life of the project and not preventable by practical means, the hazard should be avoided. Developments should be built sufficiently far away from the threat that they will not be affected by potential offsite failures. Proposed development areas at or near the base of unstable slopes should be avoided and relocated to areas where stabilization is feasible;
2. **Reduce the Hazard to an Acceptable Level:** Several techniques can be used to increase the factor of safety to a level that is acceptable to the local permitting agency. The commonly

accepted factor of safety for slopes is > 1.5 for static and > 1.1 for dynamic loads. For liquefaction hazards, reducing surface settlement to $1''/30'$ is generally accepted, depending upon the nature of the building (Boone, 1996); and,

3. **Accommodate the hazard:** Where conditions exist that will cause some measurable amount of strain, engineering techniques based on performance can be used to accommodate the stress. Reducing the hazard may not ensure that the project will remain stable indefinitely; however, the continued success of mitigation often depends on timely inspection, maintenance and ongoing repair.

The objectives of this chapter are to a) Describe and recommend geotechnical techniques that lessen the earthquake-induced landslide and liquefaction hazards found within zones of required investigation, and b) Promote the consistent, statewide use of these mitigation techniques for projects subject to the Seismic Hazards Mapping Act.

Basic Considerations

In general, only removal or densification of all potentially unstable soils, or permanently lowering the groundwater can fully eliminate landslide and liquefaction hazards. However, foundation treatments and other structural methods may be used to reduce excessive settlement. For example, in areas where liquefaction may cause displacements, designing the foundation to withstand the expected displacement will significantly reduce future damages from liquefaction.

Permitting agencies may set suitable levels of protection prior to project approval (CGOA, 2005). However, these recommended levels may vary among agencies. One of the methods of standardizing requirements is to set statewide, allowable limits based on project performance. Where the structural load is light in weight, such as in typical single-family residential houses, a post-tensioned slab foundation system may be used to accommodate differential settlements up to one inch. The reinforced slab gives the slab sufficient rigidity to span voids that may develop due to differential settlements. For heavier buildings with heavier loads and a relatively uniform mass distribution, a thicker mat foundation is feasible. Buildings supported on continuous spread footings with isolated footings can be interconnected with grade beams to improve support.

Slope stability depends upon the slope geometry, driving forces, distribution of earth materials, ground water conditions, material densities, and material strengths (Varnes, 1978). Stability analyses result in a factor of safety, which is defined as the sum of the driving forces minus resisting forces. A slope is considered to be at the point of failure when the factor of safety equals one, i.e., when the shear strength exactly balances the shear stress induced by gravity. A slope has reserve strength when FS is greater than one. Historically, most jurisdictions require a slope stability analysis for cut, fill and natural slopes that have a slope gradient steeper than two horizontal to one vertical (2:1) or slopes that possess adverse geologic conditions, show prior landslide activity or are unstable under seismic loading conditions.

It is common at construction sites to use multiple methods to achieve adequate mitigation. The geotechnical techniques used depending upon the geotechnical hazards present and the types of structures proposed. The intent of the Seismic Hazards Mapping Act is to balance development costs and safety by requiring adequate site analysis and promoting appropriate mitigation techniques for the conditions found.

Mitigation of Landslide Hazards

The two types of grading commonly used to mitigate landslide hazards in California are “hillside” construction of building pads and “flat land” construction of engineered mats. These grading techniques are used to prepare level building lots, to provide a means of removing and replacing poor soils, to stabilize landslides, to assure proper drainage, minimize liquefaction and reduce differential settlement. The grading practices used to minimize slope instability on hillside projects are better developed than the grading practices used to minimize liquefaction on flatland projects. This is partially because rainfall-induced slope failures occur more often and are more widespread than earthquake-induced liquefaction failures. As a result, most permitting agencies developed guidelines for hillside grading well before mitigation was required under the Seismic Hazards Mapping Act. Four general categories of earthquake-induced slope failures are discussed below; rotational landslides, fill displacements, soil flows and rock falls.

Rotational Landslides

Rotational slides are characterized by a somewhat cohesive slide mass that rotates along a relatively deep failure plane. The rotational failure occurs at the base of the landslide along one discrete, or several relatively thin zones, of weakness. The principal failure mechanism is the loss of shear strength at depth along a rupture surface that results in sliding of the rock or soil mass within the slope.

Hazard Description

Complex features that develop on a slide mass include rotated blocks and extensional grabens near the head scarp, compressive ridges in the main body and debris or earth flows near the toe of the slide. These features can stress buildings on the slide mass. They also provide conduits for infiltration of water, which can increase the driving force of the failed material. If naturally occurring landslides are not sufficiently removed before engineered fill is placed, movement below a fill can be reactivated along the pre-existing failure plane and transmitted to the surface, causing settlement of building pads (Rogers, 1992). Strong earthquake shaking can cause this type of slope failure even in properly engineered fill that has been placed above a graded surface. Material strength changes, water content and settlement that occur through time in the fill can add to the failure potential.

Recommended Mitigation

Appropriate mitigation depends upon accurate and complete hazard recognition. The standard practice for ensuring stability against earthquake-induced rotational slides is to adequately explore all potential instabilities and treat them during the rough grading phase before construction begins. The overall grading goal is to reduce the driving forces in the upper parts of the slide mass and to increase the resisting forces in the toe area of the slope by providing shear keys or buttresses in the subsurface. Most deep fills need to have water diverted from the fill to enhance stability. Sub-drain galleries are used to prevent pore water pressure build-up in constructed engineered fill. Surface drains, hydra-augered sub-drains, tarps, ditches and grading are used to help stabilize the slide mass once failure has occurred. During re-construction of a landslide mass, the standard practice for stabilizing is to perform all of the following:

1. Remove the unstable soil and rock from the existing slide;
2. Regrade the slide mass to a more stable configuration;
3. Scarify the failure plane to form a bond between the in-place soils and the fill soils;
4. Install sub-drains to relieve hydrostatic pressure at the base of the fill; and,
5. Apply slope stabilization methods such as key trenches, buttress fills or crib walls as appropriate.

Grading can totally remove the landslide or flatten the surface slope angle by ‘laying back’ the slope face to a stable angle. Grading is also used to reduce the weight of the slide mass and direct water away from the surface to prevent infiltration. In some cases, lightweight fill materials will be used to lighten the weight at the head of the slide and layered geofabrics used during recompaction will be used to increase shear strength in the body of the slide. A buttress fill constructed at the toe of the slide will help support the upslope portion of the mass. Buttress fills have a wide base, typically ranging from one third to almost the full height of the slope being buttressed. Fill should be compacted to a minimum of 90 percent of the maximum density as per ASTM D1557 (AASHTO, 2002).

Smaller scale slope failures can occur on graded benches. This type of failure is often a function of improper erosion control measures and lack of drain upkeep. The common mitigation for this type of failure is to prepare adequately compacted fill and stable cut slopes, and ensure an adequate setback. Although considerable engineering and geologic judgment is required for each site, the general geometry for setbacks has been a part of the Uniform Building Code for years (ICBO, 2001). The general heights and distances for compacted fill are provided by code and considered typical regulatory minimums for graded lots. However as development has moved into steeper slopes, the geometry is an inadequate parameter measure of slope safety. Stability on difficult lots depends upon material strength, compaction, and aspect of bedding and other discontinuities rather than geometry.

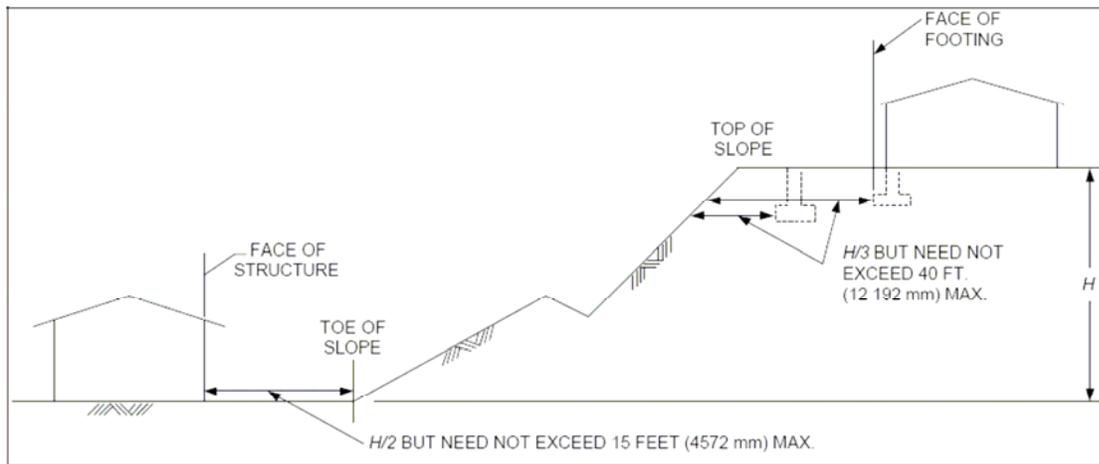


Figure 2. Minimum Setbacks established by the California Building Code.

Fill Displacements

Fill displacement failures are displacements that commonly occur at depth beneath a deep fill or between the natural ground at the edge of a cut slope and the engineered fill of the bench or pad. This type of failure is caused by static gravity force, and results in gradual settlement over time or accelerated settlement in response to dynamic earthquake forces. If there are significant differences, bedrock will be impacted differently than fill material during ground shaking because of the dissimilar material properties.

Hazard Description

The most common fill displacement hazard is differential settlement, which can severely damage building foundations, roads and lifelines. Cut-and-Fill transitions are a special case of differential settlement (Stewart and others, 2001). This hazard impacts side hill benches that have been cut for house pads and roads built on fill. Excessive settlement and fissures can also occur in deep canyons that have been filled in with imported material. Because fills are typically less dense than the underlying materials, much of the adjustment to settlement takes place either at the fill boundary with the natural canyon wall or near the center of the fill where material is the thickest.

Recommended Mitigation

The standard practice for stabilizing settlement failures at cut-fill transitions is to over-excavate during construction and grade the bedrock surface in multiple steps to provide a gradual slope transition (Figure 3). Fill should be compacted to a minimum of 90 percent of the maximum density as per ASTM D1557. Scarification provides a bond between the fill material and the underlying native rock. The overall grading goal is to minimize the difference in bearing capacity across the cut-fill boundary.

Although the California Building Code may be adequate for homogeneous engineered fill, the suggested geometry does not adequately consider bedding plane weaknesses, weathering, hydrostatic pressures or shear strength of the material.

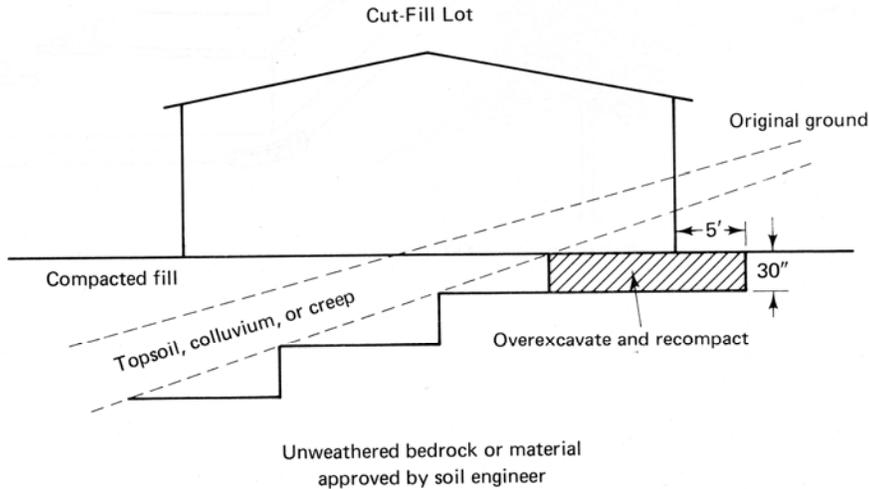


Figure 3. The typical mitigation for a Cut-and-Fill lot is to construct a gradual “stepped” transition between bedrock and fill, overexcavating unstable soils and recompacting suitable fill beneath the footprint (Stewart and others, 2001).

A common grading solution for excess settlement in deep canyon fill failures is to flatten the canyon walls during grading and compact the fill in “zones” at different depths and positions in the fill (Noorany, 1997). The final density of deep fills is determined by the water content, relative compaction and dry density of each fill zone during placement. The gradual reduction in soil strength needs to be considered in assessing the long-term performance of fills. This reduction often results from increased pore pressure due to irrigation, precipitation and earthquake-induced settlement.

The long-term stability of cut and fill slopes and deep canyon fills requires drainage and erosion control measures and ongoing maintenance. That responsibility must be transferred through the life of the development. These maintenance activities include the following tasks:

1. Establish erosion-resistant vegetation on the slope face;
2. Maintain irrigation systems so they do not introduce excess water into the fill;
3. Ensure that sub-drains are kept open and control pore pressures at the base;
4. Keep surface drains in working order and discharging to acceptable outflows;
5. Control surface drainage, especially on building pads located above slopes; and,
6. Repair erosion failures and surficial slope failures before they progress.

Soil Flows

Soil flows/slips are generic terms for shallow disrupted slides composed of loose combinations of soil, surficial deposits, rock fragments, weathered rock and vegetation. The principal failure mechanism in this type of flow is fluidization of the soil mass, caused by a reduction in shear strength due to increased pore water pressure during rain. This type of failure can be triggered by additional forces acting on hill slope materials during strong earthquake shaking, which can also induce dynamic compaction and increase pore water pressure, further weakening the slide mass. Soil flows can be subdivided by grain size into debris flows where the material is coarse-grained and earth flows where fine-grained. The geomorphic character, speed and travel distance of a soil flow is dependant on the particle size, slope and water content within the slide mass. Debris flows form steep, unvegetated scars in the head region and irregular, hummocky deposits at the toe. They most commonly occur on slopes greater than 65 percent. The 1994 Northridge Earthquake in southern California triggered more than 11,000 shallow disrupted slides (Harp & Jibson, 1995).

Hazard Description

Earth flows are characteristically slow moving and may continue to move for a period of days to weeks after initiating. The main hazard from flows occurs where they impact projects in the downslope or runout area. They have a main slide plane at their base and, in larger landslides of this type, sliding occurs on many discontinuous shear surfaces throughout the landslide mass, leading to a surface expression that resembles the flow of a viscous liquid.

Debris flows may initiate as slides, but almost immediately break up and turn into flows. They are typically triggered by periods of prolonged rainfall following a period of less intense precipitation. They can move very rapidly and travel relatively long distances, making them a significant threat to life and property. Individual debris flows are typically small in areal extent and their deposits are relatively thin. They form run-out deposits on the flatter ground below the failure. The extent of runout is based on volume and speed of the debris mass. Loose material that has accumulated in swales on steep slopes provides the flow material. Flows can also occur on the outer slope of engineered fill faces where saturated surface soils lay above more highly compacted engineered fill.

Recommended Mitigation

Because this type of hazard usually develops in distinct chutes, the main mitigation is to either avoid the hazardous zone or deflect the flow material (Hollingsworth and Kovacs, 1981). The most common solution is to provide an adequate setback from the runout zone. Grading solutions include removing excess material from the upslope swales, reshape the gully profile to reduce the driving forces, lowering the slope gradient and restricting water inflow into the soil mass. Offsite flows can be mitigated using catchment basins, protective structures such as embankments, diversion or barrier walls and by requiring setback distances. The most effective

measure to protect structures against earth flows and debris flows is to accurately define the potential failure area and require a setback from the runout path.

Rock Falls & Topples

Rock falls and topples consist of weakly cemented, loose or intensely fractured and weathered material on slopes. The falling blocks are created by sets of fractures or intact blocks in a weak matrix. Natural fracturing patterns and incipient failure planes determine block sizes. The principal failure mechanism in this type of failure is loss of tensile strength on very steep slopes. This loss of tensile strength is commonly accentuated or triggered by infiltration of water, freeze-thaw cycles or strong earthquake ground shaking.

Hazard Description

Rock falls commonly occur on high angle cut slopes, ledges, steep slopes, and in particular, highway and railroad cuts where slopes have been undercut either during construction or over steepened by progressive removal of small slope failures and ongoing maintenance. They pose a substantial hazard to vehicles along roadways and to structures downslope at the base of canyons.

Recommended Mitigation

The most common mitigation on steep slopes with large blocks and well-defined discontinuities is to increase the resisting force by pinning individual blocks and slide masses with rock bolts and anchors. More highly fractured rock masses can be contained by installing reinforced caissons, covering the slope with wire mesh or by scaling overhanging rock from the slope face. Another common solution is to separate the structure from the hazard with an adequate setback, build a graded berm to divert or adequately contain the material. In more homogeneous, fine-grained material, grading can be used to decrease the steepness of the slope and reduce the driving forces. These soil/weathered rock mixtures can also be pinned using soil nail or sprayed with gunite to stabilize the slope faces. In some cases, the unstable material must be removed from the slope face using mechanical means or hand labor.

Category	Recommended Mitigation Method	Important Considerations
Initial Grading	Excavate to competent material and and replace with engineered fill.	Most commonly used method of large-scale landslide treatment during initial grading. Drainage galleries, benching, compaction and scarification provided by design. Requires sufficient stability to resist maximum allowable seismic "triggering" displacement.
Remedial Grading	Reconfigure the mass to a more stable configuration at a lower slope angle.	Grade to reduce the slope geometry: remove material from the head and add counterweight material and key trenches in the toe area. Usually done in conjunction with dewatering. Area must be accessible to equipment and a disposal site is required for excavated material.
Engineered Fill	Construct cut and fill benches to provide level building sites, roads and utilities.	Overexcavate all transition pads or avoid building over a cut-fill transition contact to reduce uneven seismic ground response.
De-watering and Drainage	Prevent "loading" of natural or engineered slopes.	Reduce water content by grading, draining or pumping water to surface ditches. French drains and dewatering wells need to be routed to a stable drain outlet.
Slope Reinforcement	Construct buttress fill and compacted earth or rock berm at the toe of landslides.	Support the toe of a slope with properly engineered fill that is keyed into competent material below potential slip circles or adverse bedding.
Internal Slope Strengthening	Install rock bolts and/or soil nails to bind material together.	Effectiveness depends upon the grain size and character of the material and anchoring. May be used with gunite to strengthen slope face.
External Retaining Structures	Build gravity and cantilliever structures.	Must have sufficient mass or angular resistance to overcome the overturning earth pressures
Internal Retaining Structures	Install pilings and/or cassions.	Piling must be founded well below the potential slide plane and close spaced or tied together with grade beams.
Avoidance	Require the use of setbacks and deflection barriers.	Avoid the runout path, install a flow deflection barrier, provide and maintain upslope debris basins and clean out colluvial hollows.
Containment	Cover slope face with wire netting, may be used in conjunction with grouting or shotcret to increase strength.	The common treatment for rock falls and topples is to install wire netting on the rock face and barriers at the slope base or remove loose material from the face of slope by mechanical means.

Table 1: Recommended Landslide Mitigation Techniques (Modified from Popescu, 2001).

Mitigation of Liquefaction Hazards

There are two general levels of liquefaction hazards; large-scale displacement, and more localized failures including the loss of bearing strength, differential settlement and small-scale flows and lateral spreads. Areas with a potential for large-scale displacements will often require multiple methods of remediation to protect against liquefaction. Treating the site with soil improvement methods can often mitigate smaller scale ground deformations. These methods help reduce future settlements due to the static forces created by building loads and the dynamic forces from earthquake-imposed stress. With improvement, safe construction can be accomplished on marginal soils; i.e., loose sand, unconsolidated fill, collapsible soil, expandable clay and mine spoils. The following discussion illustrates some of the current techniques used to mitigate common liquefaction hazards like large-scale displacements, loss of bearing strength, settlement, and lateral spreads.

Large-Scale Displacements

Youd (1989), citing data from Japan, suggests that structural mitigation may be acceptable where displacements of less than one foot horizontal and less than four inches vertical are predicted. Therefore, for this paper, large-scale ground displacements are defined as those that exceed 1-3 feet horizontally and 4-6 inches vertically.

Large Spreads and Flows

Lateral spreads most often occur on gentle sloping ground, but can also occur on flat-lying terrain adjacent to a free face where an underlying layer liquefies in response to earthquake ground motions. The material moves on laterally continuous layers of loose, saturated gravel, sand, silt or sensitive clay. This type of failure commonly occurs on flood plains, river channel alluvium and in artificial fill on slopes as gentle as 0.3 degrees (Keefer, 1984), and can produce displacements of a few inches to tens of feet. Materials range in composition from clay and silt to fine-grained sand and may include sandy gravel.

Hazard Description

The main type of destructive movement in large-scale displacements is lateral extension accompanied by shearing or tensile fracturing. In response, the overlying material may break into tilted blocks and be rafted a few meters to tens of meters. In extreme cases, ground movements can disrupt structures, roads and utilities such as in Turnagain Heights during the 1964 Alaska earthquake. One of the problems with large-scale lateral movement is the lack of hazard recognition during development. The potential for failure can be overlooked because of subsurface data and the widespread, diffuse nature of the liquefiable layers.

Recommended Mitigation

Only removal and densification of liquefiable soils, or permanently lowering the groundwater by dewatering, can fully eliminate liquefaction hazards. Large-scale displacements may be mitigated to some degree using the following techniques:

1. Edge containment structures that provide lateral support;
2. Densification of liquefiable soils to reduce liquefaction potential;
3. Modification of site geometry to reduce the risk of movement; and
4. Drainage to lower groundwater below the level of the liquefiable soils.

A common method used to limit large-scale displacements near harbors and marinas is to restrict movement using edge containment, coupled with deep dynamic compaction to densify and compact the subsurface soils. The lateral extent where improvement is required is related to the bearing capacity of the soil and the distance that excess pore pressures will propagate beyond the structure footprint. Studies by Iai (1988) indicate that, in the presence of liquefiable clean sands, an area of softening can occur a distance beyond the foundation that is up to two-thirds the thickness of the liquefiable layer. For example, a sixty-foot thick vertical layer can cause failure up to 40 feet laterally from the edge of improved ground. Edge containment structures may include berms, dikes, sea walls, retaining structures or compacted soil zones. In extreme cases, additional deep foundation support may be required to ensure building safety.

Localized Failures

Localized ground failures are smaller scale displacements that can often be mitigated by treating the site with soil improvement methods. There are several published articles with additional information on the success of specific soil improvement techniques (Hayden and Baez, 1994). Suitable mitigation alternatives may include one or more of the following:

1. Excavation and removal or re-compaction of potentially liquefiable soils;
2. Soil densification or other types of in-situ ground modifications;
3. Deep foundations that have been designed to accommodate liquefaction effects; and,
4. Reinforced shallow foundations and improved structural design to withstand predicted vertical and lateral ground displacements.

Three types of deformations cause most of the damage in small-scale failures; loss of bearing strength, differential settlement and soil spreads and flows.

Loss of Bearing Strength

Loss of bearing strength due to liquefaction can occur in saturated clean sand under strong earthquake loading conditions. This type of failure often occurs in natural deposits of clean sands that have interbedded lenses of finer-grained sediments at some depth below the ground surface. In addition, overburden pressure tends to increase density with depth. The fill deposits most susceptible to liquefaction and excessive settlement are thick accumulations of clean, cohesionless sand that are saturated and do not strengthen substantially with depth.

Hazard Description

In structures with supporting columns, loss of bearing strength and soil-structure-soil interaction affects such as the “punching” phenomena caused by the rocking motions of structures during earthquakes. This motion can lead to tilting and uneven settlement of buildings and damage to foundations from differential settlements.

Recommended Mitigation

The most widely used technique to mitigate loss of bearing strength is in-situ densification of liquefiable soils using vibro-compaction, vibro-replacement, deep dynamic compaction, and compaction grouting (Hayden and Baez, 1994). Vibro-compaction and vibro-replacement techniques use similar equipment, but use different backfill material to achieve densification of soils at depth. In vibro-compaction a sand backfill is generally used, whereas in vibro-replacement stone is used as backfill material. Vibro-compaction is generally effective if the soils to be densified are sands containing less than approximately 10 percent fine-grained material passing the No. 200 sieve. Vibro-replacement is generally effective in soils containing less than 15 to 20% fines. However, even non-plastic sandy silts can be densified by a combination of vibro-replacement and vertical band (wick) drains.

Differential Settlement

Differential settlement is a localized loss of support under the footprint or across the span of a building. It commonly occurs in interbedded sediments at alluvial sites. The variable lensing character of the material makes the extent and thickness of the liquefiable layers; and therefore, the amount of settlement, hard to predict. Building sites require sufficient test borings and adequate testing to accurately determine the settlement potential.

Hazard Description

A major problem with lensing is the potential for uneven or differential settlement and associated fissuring across the footprint of a building which, when transmitted through the foundation, can cause structural damage. This type of failure coupled with lateral spreading is also of particular concern to the stability of dikes and levees along rivers and deltas, and to bulkhead walls and other port and harbor structures. Grading solutions include excavating and re-compacting the subsurface with properly engineered material and techniques. Internal clay core zoning and

over-top protection are important additions to the mitigation program for levees, berms and dams.

Recommended Mitigation

The typical grading solution to this type of failure is to estimate the amount of potential vertical settlement, then design and construct a mat of compacted fill that is thick enough to form a uniform bearing surface. Designing the thickness of engineered mats is intended to dissipate different amounts of displacement (Martin and Lew, 1999). The main technique used is to remove and re-compact a soil mat to give the foundation a more stable base. A variation of the technique is to actually construct the engineered mat above the existing ground level instead of excavating below grade. In general, the thicker the mat, the greater amount of settlement it can accommodate. A raised mat has the added impact of providing greater separation from a shallow water table

Piles or caissons extending to non-liquefiable soil or bedrock below the potentially liquefiable soils may be feasible. Such designs should take into account the possible down drag forces on the foundation elements due to settlement within the liquefiable upper soils. Because there may be a considerable loss of lateral soil stiffness and capacity during shaking, the piles or caissons will have to transmit the lateral loads to the deeper supporting soils.

It should be recognized that structural mitigation might not reduce the potential of the soils to liquefy during an earthquake. There will remain some risk that the structure could still suffer damage and may not be useable if liquefaction occurs. Utilities and lifeline services provided from outside the structure could still suffer disruption unless mitigation measures are employed that would account for the soil deformations that could occur between the structure and the supporting soils. Repair and remedial work should be anticipated after a liquefaction event if structural mitigation is used.

Vibro-compaction: All soil improvement can be thought of as constructing an engineered mat with denser properties than the original soil. Densification methods include deep dynamic compaction, compaction grouting, permeation grouting, jet grouting, and deep soil mixing. Design aids are available to assist in selecting suitable methods of mitigation. For example, Figure 4 indicates method suitability as a function of grain size for different soils (Hayward Baker Inc., 1997).

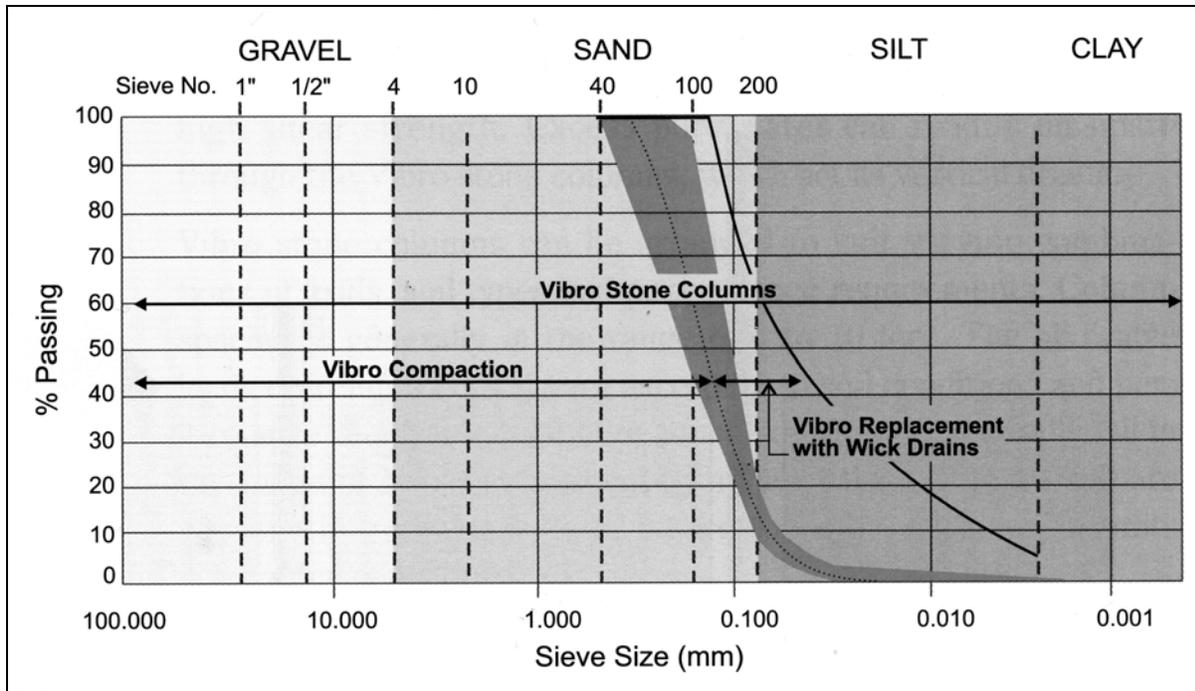


Figure 4. Gradation chart showing the range for some of the preferred soil treatment methods based on different soil contents (Hayward Baker, 1997).

Deep dynamic compaction: Deep dynamic compaction programs are used to reconstitute liquefiable soils to a denser condition using weights of 10 to 30 tons dropped from heights of about 50 to 120 feet. Free-fall impact energy is controlled by selecting the weight, drop height, number of drops per point and the grid spacing. The major limitations of the method are vibrations, flying matter, and noise. For these reasons, work often requires 100 to 200 feet clearance from adjacent occupied buildings or sensitive structures.

In general, treatment depths of up to 35 feet may be achievable in granular soils. If hard pans and saturated cohesive soils are present or the groundwater table is within 3 to 5 feet of the surface, a granular layer is often needed to limit the loss of impact energy and transfer the forces to greater depths. Pore water pressures of an area recently treated should be allowed to dissipate before secondary treatments are implemented.

Compaction grouting: uses low slump, mortar-type grout pumped under pressure to densify loose soils by displacement. Grout pipes are installed in a grid pattern that usually ranges from 5 to 9 feet on centers. The use of primary spacing patterns with secondary or tertiary intermediate patterns in-filled later is effective to achieve difficult densification criteria. Grouting volumes can typically range from 3 to 12 percent of the treated soil volume in granular soils, although volumes up to 20 percent have been reported for extremely loose sands or silty soils. Inadequate compaction is likely to occur when there is less than 8 to 10 feet of overburden to provide vertical confinement.

Permeation grouting: involves the injection of low viscosity liquid grout into the pore spaces of granular soils. The base material is typically sodium silicate or microfine cements. With

successful penetration and setting of the grout, a liquefiable soil with less than approximately 12 to 15 percent fine-grained fraction becomes a hardened mass.

Jet grouting: forms cylindrical or panel shapes of hardened soils to replace liquefiable, sensitive, or permeable soils with soil-cement having strengths up to 2,500 psi. The method relies on water pressure up to 7,000 psi at the nozzle to cut soils, mix the cement slurry in place and lift spoils to the surface. Control of the drill rotation and pull rates allows treatment of various types of soils. Lightweight drill systems can be used in confined spaces such as inside existing buildings that are found to be at risk of liquefaction after construction.

Deep soil-mixing: is a technique involving mixing of cement using a hollow-stem auger and paddle arrangement. Augers up to 3 feet or more in diameter are used to mix to depths of 100 feet or more. As the augers are advanced into the soil, the hollow stems are used as conduits to pump grout and inject into the soil at the tip. Confining cells are created with the process as the augers are worked in overlapping configurations to form walls. Liquefaction is controlled by redistributing shear stresses from soils within the confining cells to the walls. As with jet grouting, treatment of the full range of liquefiable soils is possible and shear strengths of 2,500 psi or more can be achieved even in silty soils.

These site remediation measures, coupled with properly engineered foundations such as heavily reinforced mats, post tensioned slabs, piles, etc., have performed well in recent earthquakes, demonstrating that many liquefaction-prone sites can be safely developed (Mitchell, 1995).

Spreads and Flows

Localized lateral spreads and small-scale flows are formed by the displacement of a surface layer in response to liquefaction of an underlying layer. This type of failure is dependent upon a gentle slope or a nearby “free face” or open area that will allow the displacement.

Hazard Description

The most common type of engineered fill that is composed of clean sand is hydraulic fill where material is placed behind a cut-off wall or bulkhead. In early developments, this technique was used near lagoons and shorelines to fill in marshland for constructing projects. It was assumed that the natural consolidation and drying process would densify the sand and provide adequate support for overlying structures. These days it is hardly ever done without some form of artificial densification.

Recommended Mitigation

The key to mitigating localized lateral spreads is to require an adequate setback from an open face or sloping ground. If the distance and geometry is restricted, then bulkhead walls or another form of retaining structure must be installed. For smaller-scale, liquefaction-induced settlements (less than 2” total), a wide range of techniques is used for site-specific problems. They may range from removal and re-compaction of shallow soils to support concrete slabs and perimeter spread footings to deep, drilled pier foundations and structural strengthening.

Category	Recommended Mitigation Method	Important Considerations
Engineered Fill	Construct a thickened mat by removing liquefiable soil and replace it with non-liquefiable fill.	The common grading solution is to remove and compact the upper 18-24 inches below a shallow slab or spread footing to increase bearing strength and bridge minor settlement. In general, higher displacements require thicker mats.
Structural	Construct a thicker slab and/or strengthen the foundation footings.	Reinforce shallow foundations with grade beams, post-tensioned slab and construct engineered mat with or without geofabric to provide lateral support.
Piles	Support the structure with cast-in-place concrete piles.	Extremely thick liquefiable soils may rely on skin friction to provide some of the support. Surrounding ground and connections may be displaced after settlement.
Over consolidation	Pre-load the area with fill or other material to decrease the rebound effects after structural load is placed.	This is a common method for heavy buildings. Increasing the thickness of compacted sub-base material beneath concrete flatwork and asphalt surfaces will reduce minor ground settlement
Mechanical Soil Improvement	Dynamic Compaction	Soil improvement using dynamic compaction from heavy weights
Chemical Soil Improvement	Grouting or chemical replacement.	Various types of in-situ soil improvement chemicals using replacement, permeation and jet grouting.
Vibratory Soil Improvement	Deep Soil Mixing	In-situ addition of material to native material while reaming out of borehole. Technique limited to certain soil types.
Edge Containment	Stability depends upon adequate edge containment berms, dikes, sea walls and other retaining structure.	Additionally, deep foundations are often required beneath structures to provide stability.
Drainage	Either lower the water table or provide surface drainage.	Grading to modify the site geometry, permanently lower water table by pumping or gravity-fed bores. Divert water using surface drainage.
Pore water dissipation	Provide conduits to dissipate pore water pressure.	The use of gravel or prefabricated drains, installed without soil densification, is unlikely to provide pore pressure relief during strong earthquakes and may not prevent excessive settlement.

Table 2: Recommended Liquefaction Mitigation Techniques (Modified from Mitchell, 1995 and Hayward Baker, 1997).

Concluding Remarks

In 1997, the Seismic Hazards Mapping Act hazard maps began to require more rigorous investigations and peer review procedures to evaluate projects sited in zoned liquefaction and earthquake-induced landslide hazard areas. The mitigations proposed here are based on proven grading techniques that should result in lower earthquake losses due to earthquake-induced slope instability, differential settlement and loss of bearing capacity in zones of required investigation.

In general, only removal or densification of all landslide and liquefiable materials can fully eliminate potential failures. However, foundation treatments and other structural methods may be used to protect against excessive settlements. For example, in areas where liquefaction may potentially cause displacements, designing the foundation to withstand or accommodate the displacement can significantly reduce future damages. The success of mitigation depends on:

1. Thorough geologic and seismic evaluation of the proposed project and site conditions, with appropriate treatment of the conditions found;
2. Careful testing of the conditions encountered during the preliminary rough grading phase of development;
3. Continuous review and inspection during grading and fill placement;
4. Monitoring soil properties, moisture content, and relative compaction during construction; and,
5. Proper post-development maintenance of the drains, outlets and erosion control.

Considerable geotechnical judgment is needed to make appropriate recommendations for the mitigation of earthquake-induced landslide and liquefaction hazards. For complicated settings, a Registered Civil Engineer and a Certified Engineering Geologist working together should perform the review, inspection and evaluation of proposed mitigations.

The intent of the State's Seismic Hazard Map Zoning program is to ensure that a minimum level of public safety and protection is met for projects in zones of required investigation. The acceptable level of risk by State standards is one that ensures life safety in most residential and commercial structures. Hospitals, schools and other essential services buildings are held to higher levels of operability and are inspected and reviewed using different standards. None of the mitigation methods suggested here should limit or supersede additional requirements set by local jurisdictions for specific site conditions.

CHAPTER 8

GUIDELINES FOR REVIEWING SITE- INVESTIGATION REPORTS

The purpose of this chapter is to provide general guidance to regulatory agencies that have approval authority over projects and to engineering geologists and civil engineers who review reports of seismic hazard investigations. These Guidelines recognize that effective mitigation ultimately depends on the professional judgment and expertise of the developer's engineering geologist and/or civil engineer in concert with the lead agency's engineering geologist and/or civil engineer.

The required technical review is a critical part of the evaluation process of approving a project. The reviewer ensures compliance with existing laws, regulations, ordinances, codes, policies, standards, and good practice, helping to assure that significant geologic factors (hazards and geologic processes) are properly considered, and potential problems are mitigated prior to project development. Under the Seismic Hazards Mapping Act, the reviewer is responsible for determining that each seismic hazard site investigation, and the resulting report, reasonably address the geologic and soil conditions that exist at a given site. The reviewer acts on behalf of a governing agency— city, county, regional, state, or federal—not only to protect the government's interest but also to protect the interest of the community at large. Examples of the review process in a state agency are described by Stewart and others (1976). Review at the local level has been discussed by Leighton (1975), Hart and Williams (1978), Berkland (1992), and Larson (1992). Grading codes, inspections, and the review process are discussed in detail by Scullin (1983).

The Reviewer

Qualifications

CCR Title 14, Section 3724(c) states that the reviewer must be a licensed engineering geologist and/or civil engineer having competence in the field of seismic hazard evaluation and mitigation. California's Business and Professions Code limits the practice of geology and engineering to licensed geologists and engineers, respectively, thereby requiring that reviewers be licensed, or directly supervised by someone who is licensed, by the appropriate State board. Local and regional agencies may have additional requirements. Nothing in these Guidelines is intended to sanction or authorize the review of engineering geology reports by engineers or civil engineering reports by geologists.

The reviewer should be familiar with the investigative methods employed and the techniques available to these professions (see Chapters 3 through 6). The opinions and comments made by the reviewer should be competent, prudent, objective, consistent, unbiased, pragmatic, and reasonable. The reviewer should be professional and ethical. The reviewer should have a clear understanding of the criteria for approving and not approving reports. Reviews should be based on logical, defensible criteria.

Reviewers must recognize their limitations. They should be willing to ask for the opinions of others more qualified in specialty fields.

If there is clear evidence of incompetence or misrepresentation in a report, this fact should be reported to the reviewing agency or licensing board. California Civil Code Section 47 provides immunity for statements made "in the initiation or course of any other proceedings authorized by law." Courts have interpreted this section as providing immunity to letters of complaint written to provide a public agency or board, including licensing boards, with information that the public board or agency may want to investigate (see *King v. Borges*, 28 Cal. App. 3d 27 [1972]; and *Brody v. Montalbano*, 87 Cal. App. 3d 725 [1978]). Clearly, reviewers need to have the support of their agency in order to carry out these duties.

The primary purpose of the review procedure should always be kept in mind: to determine compliance with the regulations, codes, and ordinances that pertain to the development. The reviewer should demand that minimum standards are met. The mark of a good reviewer is the ability to sort out the important from the insignificant, to list appropriate requirements for compliance, and to assist the applicant and their consultants in meeting the regulations without doing the consultant's job.

Conflict of Interest

In cases where reviewers also perform geologic or engineering investigations, they should **never** be placed in the position of reviewing their own report, or that of their own agency or company.

Reviewing Reports

The Report

A report that is incomplete or poorly written should be **not** approved. The report should demonstrate that the project complies with applicable regulations, codes, and ordinances, or local functional equivalents, in order to be approved.

The reviewer performs four principal functions in the technical review:

1. Identify any known potential hazards and impacts that are not addressed in the consultant's report. The reviewer should require investigation of the potential hazards and impacts;

2. Determine that the report contains sufficient data to support and is consistent with the stated conclusions;
3. Determine that the conclusions identify the potential impact of known and reasonable anticipated geologic processes and site conditions during the lifespan of the project; and,
4. Determine that the recommendations are consistent with the conclusions and can reasonably be expected to mitigate those anticipated earthquake-related problems that could have a significant impact on the proposed development. The included recommendations also should address the need for additional geologic and engineering investigations (including any site inspections to be made as site remediation proceeds).

Report Guidelines and Standards

Investigators may save a great deal of time (and the client's money), and possibly misunderstandings, if they contact the reviewing geologist or engineer at the initiation of the investigation. Reviewers typically are familiar with the local geology and sources of information and may be able to provide additional guidance regarding their agency's expectations and review practices. Guidelines for geologic or geotechnical reports have been prepared by a number of agencies and are available to assist reviewers in their evaluation of reports (for example, CGS Notes 42, 44, 48, and 49). Distribution of copies of written policies and guidelines adopted by the agency usually alerts the applicants and consultants about procedures, report formats, and levels of investigative detail that will expedite review and approval of the project.

If a reviewer determines that a report is not in compliance with the appropriate requirements, this fact should be stated in the written record. After the reviewer is satisfied that the investigation and resulting conclusions and recommendations are reasonable and meet local requirements, approval of the project should be recommended to the reviewing agency.

Review of Submitted Reports

The review of submitted reports constitutes professional practice and should be conducted as such. The reviewer should study the available data and site conditions in order to determine whether the report is in compliance with local requirements. A field reconnaissance of the site should be conducted, preferably after the review of available stereoscopic aerial photographs, geologic maps, and reports on nearby developments.

For each report reviewed, a clear, concise, and logical written record should be developed. This review record may be as long or short as is necessary, depending upon the complexity of the project, the geology, the engineering analysis, and the quality and completeness of the reports submitted. At a minimum, the record should:

1. Identify the project, pertinent permits, applicant, consultants, reports and plans reviewed;
2. Include a clear statement of the requirements to be met by the parties involved, data required, and the plan, phase, project, or report being approved or denied;
3. Contain summaries of the reviewer's field observations, associated literature and air photo review, and oral communications with the applicant and the consultant;
4. Contain copies of any pertinent written correspondence; and,
5. The reviewer's name and license number(s), with any associated expiration dates.

The report, plans, and review record should be kept in perpetuity to document that compliance with local requirements was achieved and for reference during future development, remodeling, or rebuilding. Such records also can be a valuable resource for land-use planning and real-estate disclosure.

Report Filing Requirements

PRC Section 2697 requires cities and counties to submit one copy of each approved site-investigation report, including mitigation measures, if any, that are to be taken, to the State Geologist within 30 days of report approval. Section 2697 also requires that if a project's approval is not in accordance with the policies and criteria of the State Mining and Geology Board (CCR Title 14, Chapter 2, Division 8, Article 10), the city or county must explain the reasons for the differences in writing to the State Geologist, within 30 days of the project's approval. Reports should be sent to:

California Geological Survey
Attn: Seismic Hazard Reports
801 K Street, MS 12-31
Sacramento, CA 95814-3531

Waivers

PRC Section 2697 and CCR Title 14, Section 3725 outline the process under which lead agencies may determine that information from studies conducted on sites in the immediate vicinity may be used to waive the site-investigation report requirement. CCR Title 14, Section 3725 indicates that when a lead agency determines that "geological and geotechnical conditions at the site are such that public safety is adequately protected and no mitigation is required," it may grant a waiver. CCR Title 14, Section 3725 also requires that such a finding be based on a report presenting evaluations of sites in the immediate vicinity having similar geologic and geotechnical characteristics. Further, Section 3725 stipulates that lead agencies must review waiver requests in the same manner as it reviews site-investigation reports; thus, waiver requests must be reviewed by a licensed engineering geologist and/or civil engineer, competent in the

field of seismic hazard evaluation and mitigation. Generally, in addition to the findings of the reports that are presented in support of the waiver request, reviewers should consider:

1. The proximity of the project site to sites previously evaluated;
2. Whether the project sites previously evaluated adequately "surround" the project site to preclude the presence of stream channel deposits, historically higher water table, gently sloping ground, stream channels and other types of free faces that may present an opportunity for lateral spread failures; and,
3. Whether the supporting reports do, in fact, conclude that no hazard exists.

Waiver Filing Requirements

CCR Title 14, Section 3725 provides that "All such waivers shall be recorded with the county recorder and a separate copy, together with the report and commentary, filed with the State Geologist within 30 days of the waiver." These materials should be sent to:

California Geological Survey
Attn: Seismic Hazard Reports
801 K Street, MS 12-31
Sacramento, CA 95814-3531

Appeals

In cases where the reviewer is not able to approve a site-investigation report, or can accept it only on a conditional basis, the developer may wish to appeal the review decision. However, every effort should be made to resolve problems informally prior to making a formal appeal. Appeal procedures are often specified by a city or county ordinance or similar instrument. An appeal may be handled through existing legal procedures, such as a hearing by a County Board of Supervisors, a City Council, or a specially appointed Technical Appeals and Review Panel. Several administrators note that the Technical Appeals and Review Panel, comprised of geoscientists, engineers, and other appropriate professionals, benefits decision makers by providing additional technical expertise for especially complex and/or controversial cases. Adequate notice should be given to allow time for both sides to prepare their cases. After an appropriate hearing, the appeals decision should be made promptly and in writing as part of the permanent record.

Another way to remedy conflicts between the investigator and the reviewer is by means of a third party review. Such a review can take different paths ranging from the review of existing reports to in-depth field investigations. Third party reviews are usually done by consultants; not normally associated with the reviewing/permitting agency.

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APPENDIX A

SEISMIC HAZARDS MAPPING ACT

CALIFORNIA PUBLIC RESOURCES CODE

Division 2. Geology, Mines and Mining

CHAPTER 7.8. SEISMIC HAZARDS MAPPING

2690. This chapter shall be known and may be cited as the Seismic Hazards Mapping Act.

2691. The Legislature finds and declares all of the following:

- (a) The effects of strong ground shaking, liquefaction, landslides, or other ground failure account for approximately 95 percent of economic losses caused by an earthquake.
- (b) Areas subject to these processes during an earthquake have not been identified or mapped statewide, despite the fact that scientific techniques are available to do so.
- (c) It is necessary to identify and map seismic hazard zones in order for cities and counties to adequately prepare the safety element of their general plans and to encourage land use management policies and regulations to reduce and mitigate those hazards to protect public health and safety.

2692.

- (a) It is the intent of the Legislature to provide for a statewide seismic hazard mapping and technical advisory program to assist cities and counties in fulfilling their responsibilities for protecting the public health and safety from the effects of strong ground shaking, liquefaction, landslides, or other ground failure and other seismic hazards caused by earthquakes.
- (b) It is further the intent of the Legislature that maps and accompanying information provided pursuant to this chapter be made available to local governments for planning and development purposes.
- (c) It is further the intent of the Legislature that the Division of Mines and Geology, in implementing this chapter, shall, to the extent possible, coordinate its activities with, and use existing information generated from, the earthquake fault zones mapping program pursuant to Chapter 7.5 (commencing with Section 2621), the landslide hazard

identification program pursuant to Chapter 7.7 (commencing with Section 2670), and the inundation maps prepared pursuant to Section 8589.5 of the Government Code.

2692.1. The State Geologist may include in maps compiled pursuant to this chapter information on the potential effects of tsunami and seiche when information becomes available from other sources and the State Geologist determines the information is appropriate for use by local government. The State Geologist shall not be required to provide this information unless additional funding is provided both to make the determination and to distribute the tsunami and seiche information.

2693. As used in this chapter:

- (a) "City" and "County" includes the City and County of San Francisco.
- (b) "Geotechnical" report means a report prepared by a certified engineering geologist or a civil engineer practicing within the area of his or her competence, which identifies seismic hazards and recommends mitigation measures to reduce the risk of seismic hazard to acceptable levels.
- (c) "Mitigation" means those measures that are consistent with established practice and that will reduce seismic risk to acceptable levels.
- (d) "Project" has the same meaning as in Chapter 7.5 (commencing with Section 2621), except as follows:
 - (1) A single-family dwelling otherwise qualifying as a project may be exempted by the city or county having jurisdiction of the project.
 - (2) "Project" does not include alterations or additions to any structure within a seismic hazard zone which do not exceed either 50 percent of the value of the structure or 50 percent of the existing floor area of the structure.
- (e) "Commission" means the Seismic Safety Commission.
- (f) "Board" means the State Mining and Geology Board.

2694.

- (a) A person who is acting as an agent for a seller of real property that is located within a seismic hazard zone, as designated under this chapter, or the seller, if he or she is acting without an agent, shall disclose to any prospective purchaser the fact that the property is located within a seismic hazard zone, if the maps prepared pursuant to this chapter or the information contained in the maps are reasonably available.
- (b) In all transactions that are subject to Section 1102 of the Civil Code, the disclosure required by subdivision (a) of this section shall be provided by either of the following means:

- (1) The Local Option Real Estate Transfer Disclosure Statement as provided in Section 1102.6a of the Civil Code.
 - (2) The Natural Hazard Disclosure Statement as provided in Section 1102.6c of the Civil Code.
- (c) Disclosure is required pursuant to this section only when one of the following conditions is met:
- (1) The seller, or seller's agent, has actual knowledge that the property is within a seismic hazard zone.
 - (2) A map that includes the property has been provided to the city or county pursuant to Section 2622, and a notice has been posted at the offices of the county recorder, county assessor, and county planning agency that identifies the location of the map and any information regarding changes to the map received by the county.
- (d) If the map or accompanying information is not of sufficient accuracy or scale that a reasonable person can determine if the subject real property is included in a seismic hazard zone, the agent shall mark "Yes" on the Natural Hazard Disclosure Statement. The agent may mark "No" on the Natural Hazard Disclosure Statement if he or she attaches a report prepared pursuant to subdivision (c) of Section 1102.4 of the Civil Code that verifies the property is not in the hazard zone. Nothing in this subdivision is intended to limit or abridge any existing duty of the seller or the seller's agents to exercise reasonable care in making a determination under this subdivision.
- (e) For purposes of the disclosures required by this section, the following persons shall not be deemed agents of the seller:
- (1) Persons specified in Section 1102.11 of the Civil Code.
 - (2) Persons acting under a power of sale regulated by Section 2924 of the Civil Code.
- (f) For purposes of this section, Section 1102.13 of the Civil Code applies.
- (g) The specification of items for disclosure in this section does not limit or abridge any obligation for disclosure created by any other provision of law or that may exist in order to avoid fraud, misrepresentation, or deceit in the transfer transaction.

2695.

- (a) On or before January 1, 1992, the board, in consultation with the director and the commission, shall develop all of the following:
 - (1) Guidelines for the preparation of maps of seismic hazard zones in the state.

- (2) Priorities for mapping of seismic hazard zones. In setting priorities, the board shall take into account the following factors:
 - (a) The population affected by the seismic hazard in the event of an earthquake.
 - (b) The probability that the seismic hazard would threaten public health and safety in the event of an earthquake.
 - (c) The willingness of lead agencies and other public agencies to share the cost of mapping within their jurisdiction.
 - (d) The availability of existing information.
- (3) Policies and criteria regarding the responsibilities of cities, counties, and state agencies pursuant to this chapter. The policies and criteria shall address, but not be limited to, the following:
 - (a) Criteria for approval of a project within a seismic hazard zone, including mitigation measures.
 - (b) The contents of the geotechnical report.
 - (c) Evaluation of the geotechnical report by the lead agency.
- (4) Guidelines for evaluating seismic hazards and recommending mitigation measures.
- (5) Any necessary procedures, including, but not limited to, processing of waivers pursuant to Section 2697, to facilitate the implementation of this chapter.
- (b) In developing the policies and criteria pursuant to subdivision (a), the board shall consult with and consider the recommendations of an advisory committee, appointed by the board in consultation with the commission, composed of the following members:
 - (1) An engineering geologist registered in the state.
 - (2) A seismologist.
 - (3) A civil engineer registered in the state.
 - (4) A structural engineer registered in the state.
 - (5) A representative of city government, selected from a list submitted by the League of California Cities.
 - (6) A representative of county government, selected from a list submitted by the County Supervisors Association of California.

(7) A representative of regional government, selected from a list submitted by the Council of Governments.

(8) A representative of the insurance industry.

(9) The Insurance Commissioner

All of the members of the advisory committee shall have expertise in the field of seismic hazards or seismic safety.

(c) At least 90 days prior to adopting measures pursuant to this section, the board shall transmit or cause to be transmitted a draft of those measures to affected cities, counties, and state agencies for review and comment.

2696.

(a) The State Geologist shall compile maps identifying seismic hazard zones, consistent with the requirements of Section 2695. The maps shall be compiled in accordance with a time schedule developed by the director and based upon the provisions of Section 2695 and the level of funding available to implement this chapter.

(b) The State Geologist shall, upon completion, submit seismic hazard maps compiled pursuant to subdivision (a) to the board and all affected cities, counties, and state agencies for review and comment. Concerned jurisdictions and agencies shall submit all comments to the board for review and consideration within 90 days. Within 90 days of board review, the State Geologist shall revise the maps, as appropriate, and shall provide copies of the official maps to each state agency, city, or county, including the county recorder, having jurisdiction over lands containing an area of seismic hazard. The county recorder shall record all information transmitted as part of the public record.

(c) In order to ensure that sellers of real property and their agents are adequately informed, any county that receives an official map pursuant to this section shall post a notice within five days of receipt of the map at the office of the county recorder, county assessor, and county planning agency, identifying the location of the map and any information regarding changes to the map and the effective date of the notice.

2697.

(a) Cities and counties shall require, prior to the approval of a project located in a seismic hazard zone, a geotechnical report defining and delineating any seismic hazard. If the city or county finds that no undue hazard of this kind exists, based on information resulting from studies conducted on sites in the immediate vicinity of the project and of similar soil composition to the project site, the geotechnical report may be waived. After a report has been approved or a waiver granted, subsequent geotechnical reports shall not be required, provided that new geologic datum, or data, warranting further investigation is not recorded. Each city and county shall submit one copy of each approved geotechnical report, including the mitigation measures, if any, that are to be taken, to the State Geologist within 30 days of its approval of the report.

- (b) In meeting the requirements of this section, cities and counties shall consider the policies and criteria established pursuant to this chapter. If a project's approval is not in accordance with the policies and criteria, the city or county shall explain the reasons for the differences in writing to the State Geologist, within 30 days of the project's approval.

2698. Nothing in this chapter is intended to prevent cities and counties from establishing policies and criteria which are more strict than those established by the board.

2699. Each city and county, in preparing the safety element to its general plan pursuant to subdivision (g) of Section 65302 of the Government Code, and in adopting or revising land use planning and permitting ordinances, shall take into account the information provided in available seismic hazard maps.

2699.5

- (a) Treasury. Notwithstanding Section 13340 of the Government Code, the moneys in the fund are continuously appropriated to the division for the purposes of this chapter.
- (c) Notwithstanding Section 5001 of the Insurance Code, one-half of 1 percent of the earthquake surcharge moneys received by the California Residential Earthquake Recovery Fund in any calendar year shall be transferred to the Seismic Hazards Identification Fund for the purposes of carrying out this chapter. This subdivision shall become operative only if Assembly Bill 3913 or Senate Bill 2902 of the 1989-90 Regular Session of the Legislature is enacted and takes effect.

2699.6. This chapter shall become operative on April 1, 1991.

APPENDIX B

SEISMIC HAZARDS MAPPING REGULATIONS

CALIFORNIA CODE OF REGULATIONS

Title 14. Natural Resources

Division 2. Department of Conservation

Chapter 8. Mining and Geology

ARTICLE 10. SEISMIC HAZARDS MAPPING

3720. Purpose

These regulations shall govern the exercise of city, county and state agency responsibilities to identify and map seismic hazard zones and to mitigate seismic hazards to protect public health and safety in accordance with the provisions of Public Resources Code, Section 2690 et seq. (Seismic Hazards Mapping Act).

Authority cited: Public Resources Code Section 2695

Reference: Public Resources Code Section 2695(a)(1)and (3)-(5)

3721. Definitions

- (a) "Acceptable Level" means that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project.
- (b) "Lead Agency" means the city, county or state agency with the authority to approve projects.
- (c) "Registered civil engineer" or "certified engineering geologist" means a civil engineer or engineering geologist who is registered or certified in the State of California.

Authority cited: Public Resources Code Section 2695

Reference: Public Resources Code Sections 2690-2696.6

3722. Requirements for Mapping Seismic Hazard Zones

- (a) The Department of Conservation, Division of Mines and Geology, shall prepare one or more State-wide probabilistic ground shaking maps for a suitably defined reference soil column. One of the maps shall show ground-shaking levels, which have a 10% probability of being exceeded in 50 years. These maps shall be used with the following criteria to define seismic hazard zones:
 - (1) Amplified shaking hazard zones shall be delineated as areas where historic occurrence of amplified ground shaking, or local geological and geotechnical conditions indicate a potential for ground shaking to be amplified to a level such that mitigation as defined in Public Resources Code Section 2693(c) would be required.
 - (2) Liquefaction hazard zones shall be delineated as areas where historic occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.
 - (3) Earthquake-induced landslide hazard zones shall be delineated as areas where Holocene occurrence of landslide movement, or local slope of terrain, and geological, geotechnical and ground moisture conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.
- (b) Highest priority for mapping seismic hazard zones shall be given to areas facing urbanization or redevelopment in conjunction with the factors listed in Section 2695(a)(2)(A), (B), (C) and (D) of the Public Resources Code.

Authority cited: Public Resources Code Section 2695

Reference: Public Resources Code Section 2695(a)(1)

3723. Review of Preliminary Seismic Hazard Zones Maps

- (a) The Mining and Geology Board shall provide an opportunity for receipt of public comments and recommendations during the 90-day period for review of preliminary seismic hazard zone maps provided by the Public Resources Code Section 2696. At least one public hearing shall be scheduled for that purpose.
- (b) Following the end of the review period, the Board shall forward its comments and recommendations, with supporting data received, to the State Geologist for consideration prior to revision and official issuance of the maps.

Authority cited: Public Resources Code Section 2696

Reference: Public Resources Code Section 2696

3724. Specific Criteria for Project Approval

The following specific criteria for project approval shall apply within seismic hazard zones and shall be used by affected lead agencies in complying with the provisions of the Act:

- (a) A project shall be approved only when the nature and severity of the seismic hazards at the site have been evaluated in a geotechnical report and appropriate mitigation measures have been proposed.
- (b) The geotechnical report shall be prepared by a registered civil engineer or certified engineering geologist, having competence in the field of seismic hazard evaluation and mitigation. The geotechnical report shall contain site-specific evaluations of the seismic hazard affecting the project, and shall identify portions of the project site containing seismic hazards. The report shall also identify any known off-site seismic hazards that could adversely affect the site in the event of an earthquake. The contents of the geotechnical report shall include, but shall not be limited to, the following:
 - (1) Project description.
 - (2) A description of the geologic and geotechnical conditions at the site, including an appropriate site location map.
 - (3) Evaluation of site-specific seismic hazards based on geological and geotechnical conditions, in accordance with current standards of practice.
 - (4) Recommendations for appropriate mitigation measures as required in Section 3724(a), above.
 - (5) Name of report preparer(s), and signature(s) of a certified engineering geologist and/or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.
- (c) Prior to approving the project, the lead agency shall independently review the geotechnical report to determine the adequacy of the hazard evaluation and proposed mitigation measures and to determine the requirements of Section 3724(a), above, are satisfied. Such reviews shall be conducted by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation.

Authority cited: Public Resources Code Section 2695

Reference: Public Resources Code Section 2695(a)(3)(A), (B), and (C)

3725. Waivers of Geotechnical Report Requirements

For a specific project, the lead agency may determine that the geological and geotechnical conditions at the site are such that public safety is adequately protected and no mitigation is required. This finding shall be based on a report presenting evaluations of sites in the immediate vicinity having similar

geologic and geotechnical characteristics. The report shall be prepared by a certified engineering geologist or registered civil engineer, having competence in the field of seismic hazard evaluation and mitigation. The lead agency shall review submitted reports in the same manner as in Section 3724(c) of this article. The lead agency shall also provide a written commentary that addresses the report conclusions and the justification for applying the conclusions contained in the report to the project site. When the lead agency makes such a finding, it may waive the requirement of a geotechnical report for the project. All such waivers shall be recorded with the county recorder and a separate copy, together with the report and commentary, filed with the State Geologist within 30 days of the waiver.

Authority cited: Public Resources Code Section 2695

Reference: Public Resources Code Section 2697(a)(5)

APPENDIX C

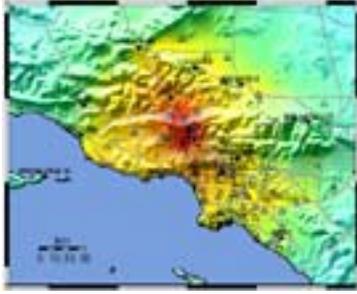
TECHNICAL TERMS AND DEFINITIONS

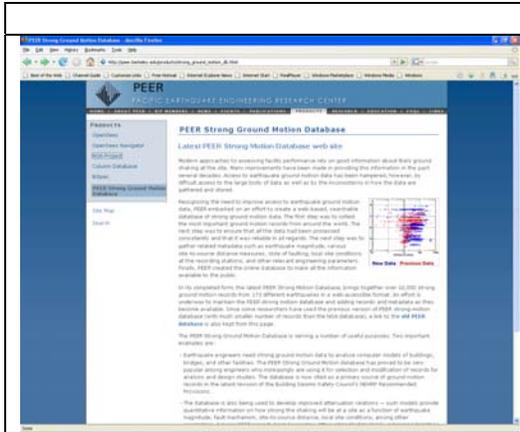
ASTM	American Society for Testing and Materials
CPT	Cone Penetration Test (ASTM D3441-94).
CSR	Cyclic stress ratio— a normalized measure of cyclic load severity imposed by an earthquake, expressed as equivalent uniform cyclic deviatoric load divided by some measure of initial effective overburden or confining stress.
CRR	The equivalent uniform cyclic stress ratio required to induce liquefaction within a given number of loading cycles [that number of cycles considered representative of the earthquake under consideration].
DSHA	Deterministic seismic hazard analysis
FS	Factor of safety— the ratio of the forces available to resist failure divided by the driving forces.
Ground Loss	Localized ground subsidence.
k	Seismic coefficient used in a pseudo-static slope stability analysis
Liquefaction	Significant loss of soil strength due to pore pressure increase.
N	Penetration resistance measured in SPT tests (blows/ft).
N₁	Normalized SPT N-value (blows/ft); corrected for overburden stress effects to the N-value which would occur if the effective overburden stress was 1.0 tons/ft ² .
(N₁)₆₀	Standardized, normalized SPT-value; corrected for both overburden stress effects and equipment and procedural effects (blows/ft).
PI	Plasticity Index; the difference between the Atterberg Liquid Limit (LL) and the Atterberg Plastic Limit (PL) for a cohesive soil. [PI(%) = LL(%) - PL(%)].
PSHA	Probabilistic seismic hazard analysis

q_c	Tip resistance measured by CPT probe (force/length ²).
q_{c,1}	Normalized CPT tip resistance (force/length ²); corrected for overburden stress effects to the q _c value which would occur if the effective overburden stress was 1.0 tons/ ft ² .
Sand Boiling	Localized ejection of soil and water to relieve excess pore pressure.
SPPV	Simple prescribed parameter values.
SPT	Standard Penetration Test (ASTM D1586-92 and ASTM D6066-96e1).
UBC	The Uniform Building Code, published by the International Conference of Building Officials (ICBO, 2001), periodically updated.

APPENDIX D

SOURCES FOR STRONG-MOTION RECORDS

Earthquake Strong Motion	
<p>It is clear that destructive earthquakes pose a continuing major threat to lives and property throughout California. Earthquake strong motion data provide information for engineers to improve earthquake resistance for buildings and other structures. The California Strong Motion Instrumentation Program (CSMIP) records the strong shaking of the ground and in structures during earthquakes for the engineering and scientific communities through a statewide network of strong motion instruments. The measured ground strong shaking is used immediately after an event to assist in emergency response by agencies like OES. Structural measurements are studied after events to analyze the performance of structures, with the goal of mitigating future earthquake impacts through improved building codes for safer, more earthquake resistant structures.</p>	
	<p>Strong-motion data for engineering applications after major earthquakes are distributed via the Internet Quick Report (IQR) and the Internet Data Report (IDR) at CGS. The IQR and the IDR are based on the design concept of the traditional Quick Report and is streamlined in an automated fashion. The release of IQR is usually accompanying with the release of ShakeMap and is for earthquakes of magnitude 4.0 or above and for events with strong-motion recordings. The release of IDR is for significant historic earthquakes.</p>
	<p>The Center for Engineering Strong Motion Data is jointly operated by the California Department of Conservation's Strong Motion Instrumentation Program (CSMIP) in cooperation with the USGS/National Strong Motion Program (NSMP). A primary goal of the Engineering Data Center as well as the other two Earthquake Data Management Centers in CISN is to provide rapid information after an earthquake, ranging from the ShakeMap to distribution of the data and calculated parameters: http://www.strongmotioncenter.org/</p>
	<p>A ShakeMap is a representation of ground shaking produced by an earthquake. The information it presents is different from the earthquake magnitude and epicenter that are released after an earthquake because ShakeMap focuses on the ground shaking produced by the earthquake, rather than the parameters describing the earthquake source. ShakeMaps are generated automatically following moderate and large earthquakes. These are preliminary ground shaking maps, normally posted within several minutes of the earthquake origin time. Under the CISN project, ShakeMaps currently are generated in both Northern and Southern California and at CGS.</p>
	<p>The California Strong Motion Instrumentation Program (CSMIP) was established in 1972 by California Legislation to obtain vital earthquake data for the engineering and scientific communities through a statewide network of strong motion instruments. When the planned network is completed, statewide coverage will ensure that strong ground motion for any moderate to larger size earthquake in the state will be recorded. Please visit: http://www.conservation.ca.gov/cgs/s mip/Pages/index.a spx</p>



The PEER Strong Motion Database is cited as a primary source of ground motion records in the latest revision of the Building Seismic Safety Council's NEHRP Recommended Provisions.

The PEER NGA Database is a select set of strong-motion records used to develop the Next Generation Attenuation Models (NGA). For latest reports on NGA models and link to the PEER Strong Motion Database, please visit:

<http://peer.berkeley.edu/nga/>

APPENDIX E

GEOLOGIC ENVIRONMENTS LIKELY TO PRODUCE EARTHQUAKE-INDUCED LANDSLIDES

Landslide Type	Type of Material	Minimum Slope	Remarks
Rock falls	Rocks weakly cemented, intensely fractured, or weathered; contain conspicuous planes of weakness dipping out of slope or contain boulders in a weak matrix.	40° 1.7:1	Particularly common near ridge crests and on spurs, ledges, artificially cut slopes, and slopes undercut by active erosion.
Rock slides	Rocks weakly cemented, intensely fractured, or weathered; contain conspicuous planes of weakness dipping out of slope or contain boulders in a weak matrix.	35° 1.4:1	Particularly common in hillside flutes and channels, on artificially cut slopes, and on slopes undercut by active erosion. Occasionally reactivate preexisting rockslide deposits.
Rock Avalanches	Rocks intensely fractured and exhibiting one of the following properties: significant weathering, planes of weakness dipping out of slope, weak cementation, or evidence of previous landsliding.	25° 2.1:1	Usually restricted to slopes of greater than 500 feet (150 m) relief that have been undercut by erosion. May be accompanied by a blast of air that can knock down trees and structures beyond the limits of the deposited debris
Rock slumps	Intensely fractured rocks, preexisting rock slump deposits, shale, and other rocks containing layers of weakly cemented or intensely weathered material.	15° 3.7:1	
Rock block slides	Rocks having conspicuous bedding planes or similar planes of weakness dipping out of slopes.	15° 3.7:1	
Soil falls	Granular soils that are slightly cemented or contain clay binder	40° 1.7:1	Particularly common on stream-banks, terrace faces, coastal bluffs, and artificially cut slopes.

Landslide Type	Type of Material	Minimum Slope	Remarks
Disrupted soil slides	Loose, unsaturated sands.	15° 3.7:1	
Soil avalanches	Loose, unsaturated sands.	25° 2.1:1	Occasionally reactivate preexisting soil avalanche deposits.
Soil slumps	Loose, partly to completely saturated sand or silt; uncompacted or poorly compacted manmade fill composed of sand, silt, or clay, preexisting soil slump deposits.	10° 11:1	Particularly common on embankments built on soft, saturated foundation materials, in hillside cut-and-fill areas, and on river and coastal flood plains.
Soil block slumps	Loose, partly or completely saturated sand or silt; uncompacted or slightly compacted manmade fill composed of sand or silt, bluffs containing horizontal or subhorizontal layers or loose, saturated sand or silt.	5° 11:1	Particularly common in areas of preexisting landslides along river and coastal flood plains, and on embankments built of soft, saturated foundation materials.
Low earth flows	Stiff, partly to completely saturated clay and preexisting earth-flow deposits.	10° 5.7:1	
Soil lateral spreads	Loose, partly or completely saturated silt or sand, uncompacted or slightly compacted manmade fill composed of sand.	0.3° 190:1	Particularly common on river and coastal flood plains, embankments built on soft, saturated foundation materials, delta margins, sand dunes, sand spits, alluvial fans, lakeshores and beaches.
Rapid soil flow	Saturated, uncompacted or slightly compacted manmade fill composed of sand or sandy silt (including hydraulic fill earth dams and tailings dams); loose, saturated granular soils.	2.3° 25:1	Includes debris flows that typically originate in hollows at heads of streams and adjacent hillsides; typically travel at tens of miles per hour or more and may cause damage miles from the source area.
Subaqueous landslides	Loose, saturated granular soils.	0.5° 110:1	Particularly common on delta margins.

Modified from Keefer (1984).