MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH

A National Center of Excellence in Advanced Technology Applications

FEDERAL HIGHWAY ADMINISTRATION

ISSN 1520-295X

Screening Guide for Rapid Assessment of Liquefaction Hazard at Highway Bridge Sites

by

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Technical Report MCEER-98-0005

June 16, 1998

This research was conducted at Brigham Young University and was supported by the Federal Highway Administration under contract number DTFH61-92-C-00106.

NOTICE

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T. L. Youd[1]

Publication Date: June 16, 1998
Submittal Date: September 21, 1995

Technical Report MCEER-98-0005

Task Number 106-E-3.1(b)

FHWA Contract Number DTFH61-92-C-00106

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MULTIDISCIPLINARY CENTER FOR EARTHQUAKE ENGINEERING RESEARCH
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Preface

The Multidisciplinary Center for Earthquake Engineering Research (MCEER) is a national center of excellence in advanced technology applications that is dedicated to the reduction of earthquake losses nationwide. Headquartered at the State University of New York at Buffalo, the Center was originally established by the National Science Foundation in 1986, as the National Center for Earthquake Engineering Research (NCEER).

Comprising a consortium of researchers from numerous disciplines and institutions throughout the United States, the Center's mission is to reduce earthquake losses through research and the application of advanced technologies that improve engineering, pre-earthquake planning and post-earthquake recovery strategies. Toward this end, the Center coordinates a nationwide program of multidisciplinary team research, education and outreach activities.

MCEER's research is conducted under the sponsorship of two major federal agencies, the National Science Foundation (NSF) and the Federal Highway Administration (FHWA), and the State of New York. Significant support is also derived from the Federal Emergency Management Agency (FEMA), other state governments, academic institutions, foreign governments and private industry.

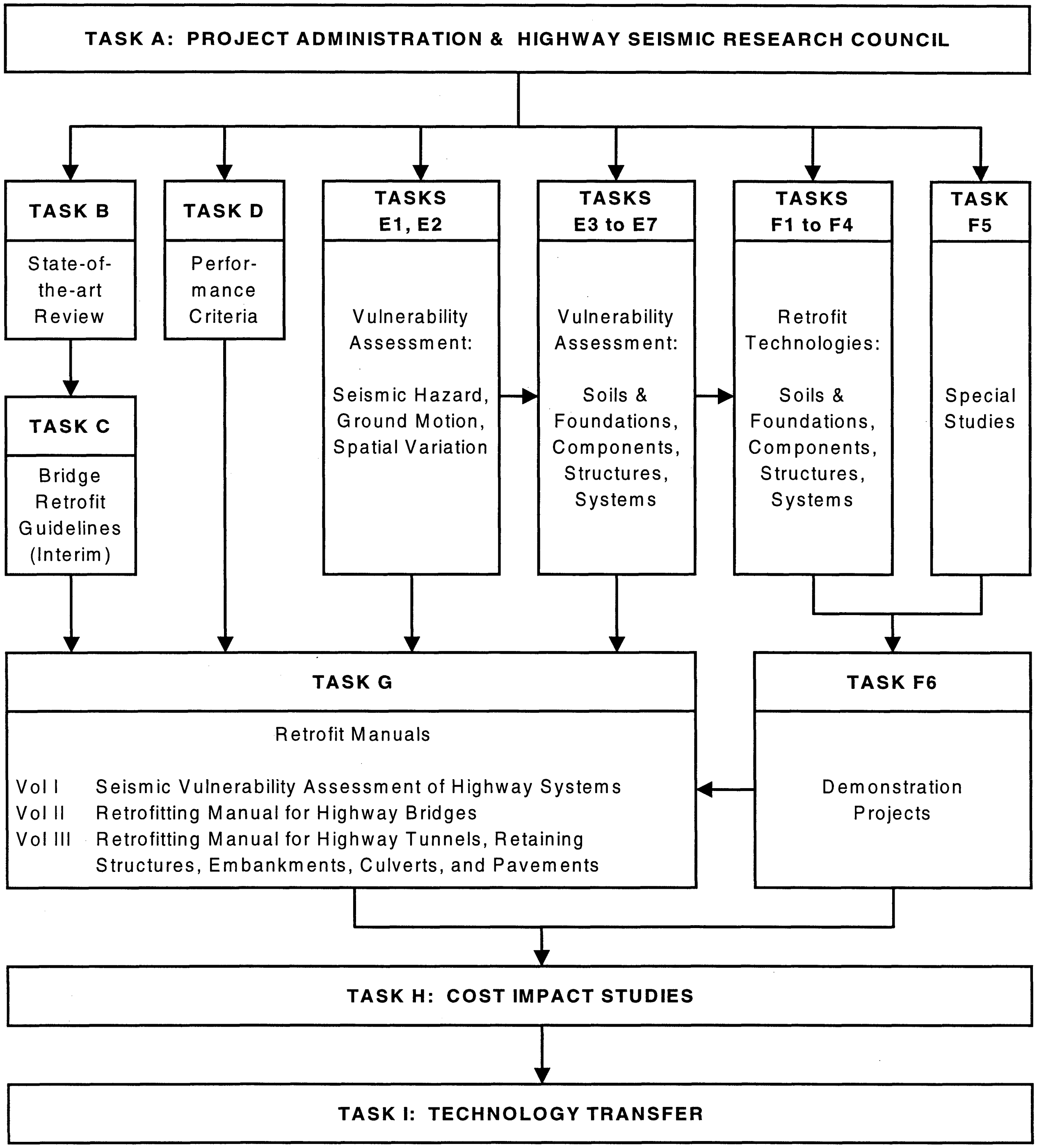
The Center's FHWA-sponsored Highway Project develops retrofit and evaluation methodologies for existing bridges and other highway structures (including tunnels, retaining structures, slopes, culverts, and pavements), and improved seismic design criteria and procedures for bridges and other highway structures. Specifically, tasks are being conducted to:

- assess the vulnerability of highway systems, structures and components;
- develop concepts for retrofitting vulnerable highway structures and components;
- develop improved design and analysis methodologies for bridges, tunnels, and retaining structures, which include consideration of soil-structure interaction mechanisms and their influence on structural response;
- review and recommend improved seismic design and performance criteria for new highway structures.

Highway Project research focuses on two distinct areas: the development of improved design criteria and philosophies for new or future highway construction, and the development of improved analysis and retrofitting methodologies for existing highway systems and structures. The research discussed in this report is a result of work conducted under the existing highway structures project, and was performed within Task 106-E-3.1(b), "Effects of Liquefaction on Vulnerability Assessment" of that project as shown in the flowchart on the following page.

The overall objective of this task was to provide procedures and guidance for highway engineers to conduct preliminary assessments of the vulnerability of existing highway structures to damage as a consequence of liquefaction-induced ground failure. This report provides a screening guide for performing a systematic evaluation of liquefaction hazard at bridge sites and a guide for prioritizing sites for further investigation or mitigation. The guide is intended for use by highway *engineers with expertise and experience in geotechnical engineering practice, but not necessarily specialized knowledge in seismic hazard evaluation. It presents a systematic application of standard criteria for assessing liquefaction, ground displacement potential, and vulnerability of bridges to damage. The screening process proceeds from least complex, least time-consuming, and least data-intensive evaluations to the more complex, time-consuming, and rigorous analyses. At each level of screening, a conservative assessment of liquefaction hazard is made. Thus, many bridge sites can be evaluated and classified as having a low liquefaction hazard with very little time or effort.*

SEISMIC VULNERABILITY OF EXISTING HIGHWAY CONSTRUCTION
FHWA Contract DTFH61-92-C-00106



ABSTRACT

Liquefaction-induced ground and foundation displacement has been a major cause of earthquake damage to bridges. For example, during the great Alaskan earthquake of March 27, 1964, liquefaction-induced lateral ground displacement inflicted structural distress to 266 railway and highway bridges, collapsing about 20 and damaging many others beyond repair. This destruction, primarily at river crossings, disrupted the surface transportation system in southern Alaska for many months after the earthquake. Numerous other occurrences of bridge damage or collapse have occurred around the world as a consequence of liquefaction. Nearly all of this damage has occurred at river or other water crossings where flat to gently sloping topography was underlain by loose, saturated, granular sediment. These types of deposits and terrain are highly vulnerable to liquefaction and lateral spread. Because of the high potential for bridge damage and consequent disruption to transportation systems, evaluation of liquefaction hazard is a major part of any assessment of seismic hazard to highway systems. As an aid to seismic hazard assessment, this report provides a "screening guide" for systematic evaluation of liquefaction hazard at bridge sites and a guide for prioritizing sites for further investigation or mitigation. This guide is intended for use by highway engineers with expertise and experience in geotechnical engineering practice, but not necessarily specialized knowledge in seismic hazard evaluation. The guide presents a systematic application of standard criteria for assessing liquefaction, ground displacement potential, and vulnerability of bridges to damage. The screening proceeds from least complex, least time-consuming, and least data-intensive evaluations to the more complex, time-consuming, and rigorous analyses. Thus, many bridge sites can be evaluated and classified as low hazard with very little time and effort. Only sites with significant hazard need to be evaluated with the more sophisticated and time consuming-procedures. At each level of screening, a conservative assessment of hazard is made. If there is clear evidence that liquefaction or damaging ground displacements are very unlikely, the site is classed as "low liquefaction hazard and low priority for further investigation," and the evaluation is complete for that site. If the available information indicates a higher hazard rating, or the data is inadequate, incomplete, or unclear, the site is classed as having possible liquefaction hazard, and the screening proceeds to the next step. If the available site information is insufficient to complete a liquefaction hazard analysis, then simplified seismic, geologic, and hydrologic criteria are used to prioritize the bridge site for further investigation.

TABLE OF CONTENTS

SECTION	TITLE	PAGE
1	**INTRODUCTION**	**1**
2	**SCREENING GUIDE**	**3**
3	**REGIONAL EVALUATIONS OF LIQUEFACTION SUSCEPTIBILITY**	**7**
3.1	Prior Evaluation of Liquefaction Hazard	7
3.1.1	Data Required	7
3.1.2	Analysis and Classification	7
3.2	Geologic Analysis	8
3.2.1	Data Required	8
3.2.2	Analysis and Classification	10
3.3	Evaluation of Seismic Hazard	10
3.3.1	Data Required	10
3.3.2	Analysis and Classification	11
3.4	Water Table Analysis	15
3.4.1	Data Required	16
3.4.2	Analysis and Classification	17
4	**SITE-SPECIFIC EVALUATIONS OF LIQUEFACTION RESISTANCE OF SANDS AND STRENGTH-LOSS IN SENSITIVE FINE-GRAINED SOILS**	**19**
4.1	Screening for Extra-Sensitive Soils	19
4.1.1	Data Required	20
4.1.2	Analysis and Classification	20
4.2	Soil Classification Evaluation	20
4.2.1	Data Required	23
4.2.2	Analysis and Classification	23
4.3	Penetration Analysis	24
4.3.1	Simplified Procedure	24
4.3.2	Standard Penetration (SPT) Resistance	26
4.3.2.1	Data Required	28

TABLE OF CONTENTS (CONT'D)

SECTION	TITLE	PAGE
4.3.2.2	Analysis and Classification	33
4.3.3	Cone Penetration (CPT) Resistance	34
4.3.3.1	Data Required	35
4.3.3.2	Analysis and Classification	39
5	**ASSESSMENT OF GROUND DISPLACEMENT HAZARD**	**41**
5.1	Embankment or Slope Instability	41
5.1.1	Data Required	41
5.1.2	Analysis and Classification	42
5.2	Analysis of Embankment or Slope Deformation	43
5.2.1	Data Required	43
5.2.2	Analysis and Classification	44
5.3	Lateral Spread Displacement	45
5.3.1	Data Required	46
5.3.2	Analysis and Classification	49
5.4	Analysis of Vertical Displacement (Ground Settlement)	49
5.4.1	Data Required	50
5.4.2	Analysis and Classification	50
5.5	Bearing Capacity Analysis	51
5.5.1	Data Required	51
5.5.2	Analysis and Classification	51
6	**PRIORITIZATION OF SITES FOR FURTHER INVESTIGATION**	**53**
6.1	Confirmed High Liquefaction Hazard	53
6.2	Confirmed Liquefaction Susceptibility, but Unknown Hazard	53
6.3	Insufficient Information to Assess Liquefaction Resistance or Strength-Loss Potential	53
6.4	Low Hazard and Low Priority for Further Investigation	54
7	**REFERENCES CITED**	**55**

LIST OF ILLUSTRATIONS

FIGURE	TITLE	PAGE
2-1	Flow Diagram Showing Steps for Liquefaction Hazard Screening	4
3-1	Map Showing Peak Horizontal Ground Accelerations	12
3-2	Map of Conterminous U.S. Showing Seismic Source Zones	13
3-3	Map of Western California Showing Seismic Source Zones	14
4-1	Chart Showing (A) UCS and (B) AASHTO Classification Systems with Stippled Region Indicative of Liquefiable Soils	22
4-2	r_d Versus Depth Curves Developed by Seed and Idriss	25
4-3	Simplified Base Curve Recommended for Calculating CRR from SPT Data, along with Empirical Liquefaction Data	27
4-4	Relationship between Moment, M_w, and Other Magnitude Scales	29
4-5	Curve Recommended for Calculating CRR from CPT Data	35
4-6	CPT-Based Soil Behavior Type Chart Proposed by Robertson	37
4-7	Thin-Layer Correction Factor, K_H, for Determining Equivalent Thick-Layer CPT Resistance	40
5-1	Empirical Relationship between Residual Shear Strength and $(N_1)_{60cs}$	42
5-2	Curves for Determining Equivalent Source Distance, R_{eq}, from Magnitude and Estimated a_{max}	47
5-3	Curves for Estimating Volumetric Strain at Liquefiable Sites	50

LIST OF TABLES

TABLE	TITLE	PAGE
3-1	Estimated Susceptibility of Sedimentary Deposits to Liquefaction during Strong Seismic Shaking	9
3-2	Minimum Earthquake Magnitudes and Peak Horizontal Ground Accelerations, with Allowance for Local Site Amplification, That Are Capable of Generating Liquefaction in Very Susceptible Natural Deposits	15
3-3	Relative Liquefaction Susceptibility as a Function of Groundwater Table Depth	16
4-1	Criteria for Assessing Liquefiability of Fine-Grained Soils	21
4-2	Comparison of Advantages and Disadvantages of Various Field Tests Used for Assessing Liquefaction Resistance	26
4-3	Corrections to SPT	33
4-4	Magnitude Scaling Factors for Liquefaction Hazard Evaluation	34
5-1	Ranges of Values for Independent Variables for Which Lateral Spread Equations Are Verified by Case-History Observations	46
5-2	Minimum Values of R for Use in Equations 5.1	47

SECTION 1
INTRODUCTION

Liquefaction-induced ground and foundation displacements have been a major cause of bridge damage during past earthquakes. For example, during the great Alaskan earthquake of March 27, 1964, liquefaction-induced lateral spread displacements generated structural distress in 266 highway and railway bridges, collapsing about 20 and damaging many others beyond repair. This destruction, primarily at river crossings, disrupted the surface transportation system in southern Alaska for many months after the earthquake (McCulloch and Bonilla, 1970; Kachadoorian, 1968; Youd, 1993a). During the 1991 Limon Province, Costa Rica earthquake, eight bridges collapsed and several others suffered severe damage as a consequence of liquefaction-induced lateral ground displacement (Youd et al., 1992; Youd, 1993a). Lateral spread of filled ground in the waterfront area of Kobe, Japan, pushed bases of several bridge piers laterally toward shipping channels, causing distress to the super structure and collapse of several spans (Buckle, 1995). Many similar incidents of bridge damage have been reported, confirming the destructive consequences of liquefaction-induced ground displacements. Nearly all of this destruction was at rivers or other water crossings where gently sloping alluvial or fill deposits liquefied and spread laterally. These types of deposits--loose, saturated, granular sediment beneath gently sloping ground or near incised channels--are highly vulnerable to liquefaction and lateral spread.

Although not as common as lateral spread, other potential causes of damage that must be considered include liquefaction-induced embankment and slope instability and deformation, ground settlement, and loss of bearing strength. Each of these modes of failure are considered in hazard assessments for highway structures.

As an aid to earthquake hazard assessment, this report provides a "screening guide" for the systematic evaluation of liquefaction resistance and damage potential for bridge sites and guidance for the prioritization of sites for further investigation and possible remediation. The screening guide is intended for use by highway engineers with expertise and experience in geotechnical engineering, but without specialized training in seismic hazard evaluation. Although the guide is specifically applied to bridges herein, the general principles could be applied to hazard assessment for most constructed works, including roadway embankments, culverts, pipelines, and buildings.

SECTION 2
SCREENING GUIDE

Liquefaction does not occur randomly in natural deposits but is limited to a rather narrow range of seismic, geologic, hydrologic, and soil environments. Taking advantage of relationships between these environments and liquefaction susceptibility, this report introduces a screening guide that highway geotechnical engineers can use to perform rapid assessments of liquefaction hazard. The guide presents a systematic application of standard criteria for assessing liquefaction susceptibility, evaluating ground displacement potential, and assessing the vulnerability of bridges to liquefaction-induced damage. The screening proceeds from least complex, time-consuming, and data-intensive evaluations to the more complex, time-consuming, and rigorous analyses. Thus many bridge sites can be evaluated and classified as low hazard with very little time and effort. Only bridges with significant hazard need to be evaluated with the more sophisticated and time-consuming procedures. At each level of screening, a conservative assessment of hazard is made. If there is clear evidence that liquefaction or damaging ground displacements are very unlikely, the site is classed as "low liquefaction hazard and low priority for further investigation," and the evaluation is complete for that bridge. If the available information indicates a likely hazard, or if the data are inadequate or incomplete, the site is classed as having possible liquefaction hazard, and the screening proceeds to the next step. If the available site information is insufficient to complete a liquefaction hazard analysis, then simplified seismic, topographic, geologic, and hydrologic criteria are used to prioritize the site for further investigation.

The screening guide is conservative; that is, at each juncture in the screening process, uncertainty is weighed on the side that liquefaction and ground failure could occur. Thus a conclusion that liquefaction and detrimental ground displacement are very unlikely is a much more certain conclusion than the converse outcome--that liquefaction and detrimental ground displacements are possible. This conservatism leads to the corollary conclusion that additional investigation is more likely to reduce the estimated liquefaction hazard than increase it.

The principal steps and logic path for the screening procedure are listed in Figure 2.1. The evaluation procedures and data requirements for each step are discussed in the following text. In assessing liquefaction hazard, the recommended procedure is to start at the top of the logic path, perform the required analyses for each step, and proceed downward until the bridge is classified into one of four categories: (1) confirmed high liquefaction and ground failure hazard--very high priority for further investigation and possible mitigation; (2) confirmed liquefaction susceptibility but unknown ground failure hazard--high priority for further investigation; (3) insufficient information to assess liquefaction susceptibility--prioritized for further investigation; or (4) low liquefaction hazard--low priority for further investigation.

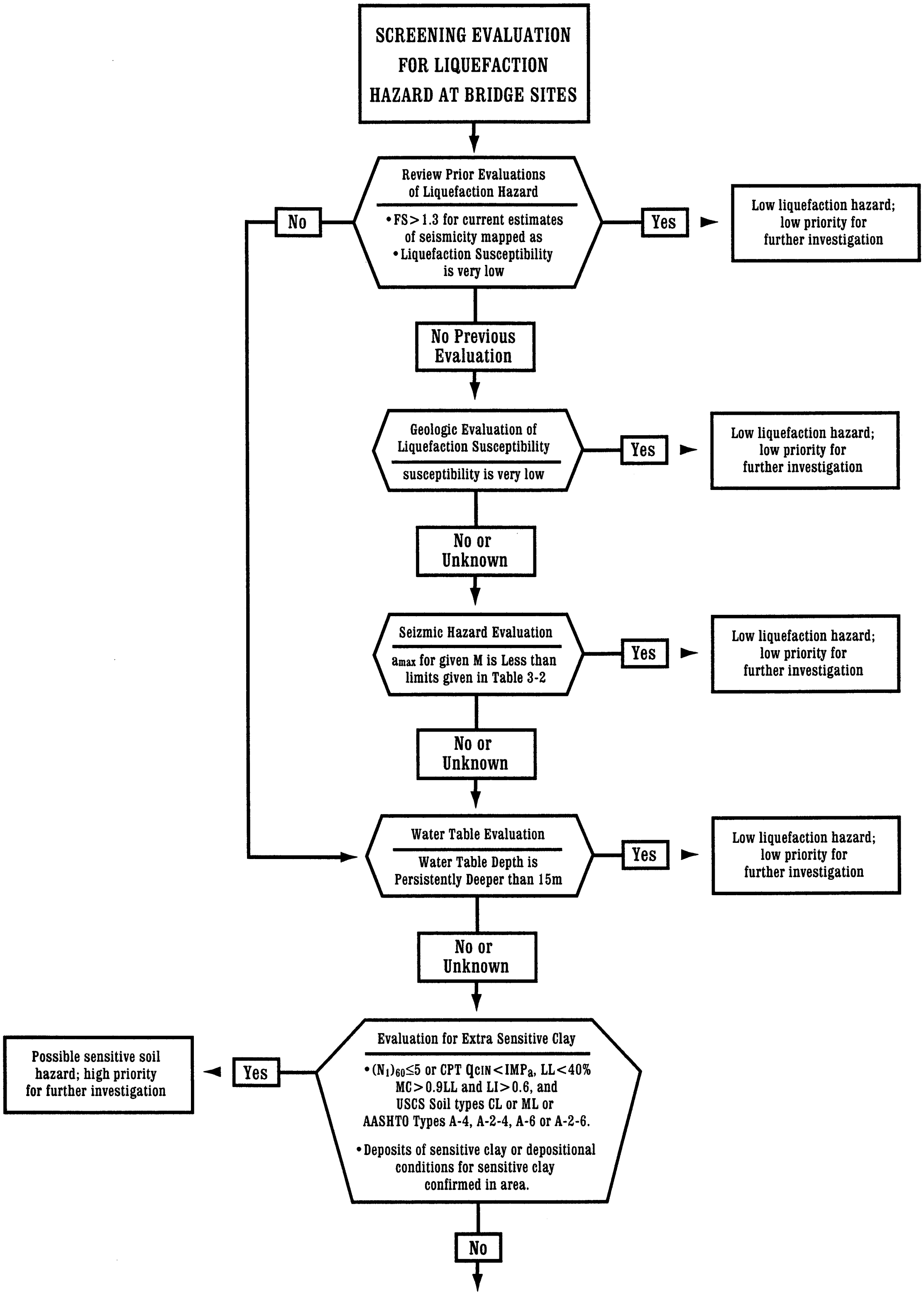


Figure 2-1. Flow Diagram Showing Steps and Criteria for Screening of Liquefaction Hazard for Highway Bridges--Part I

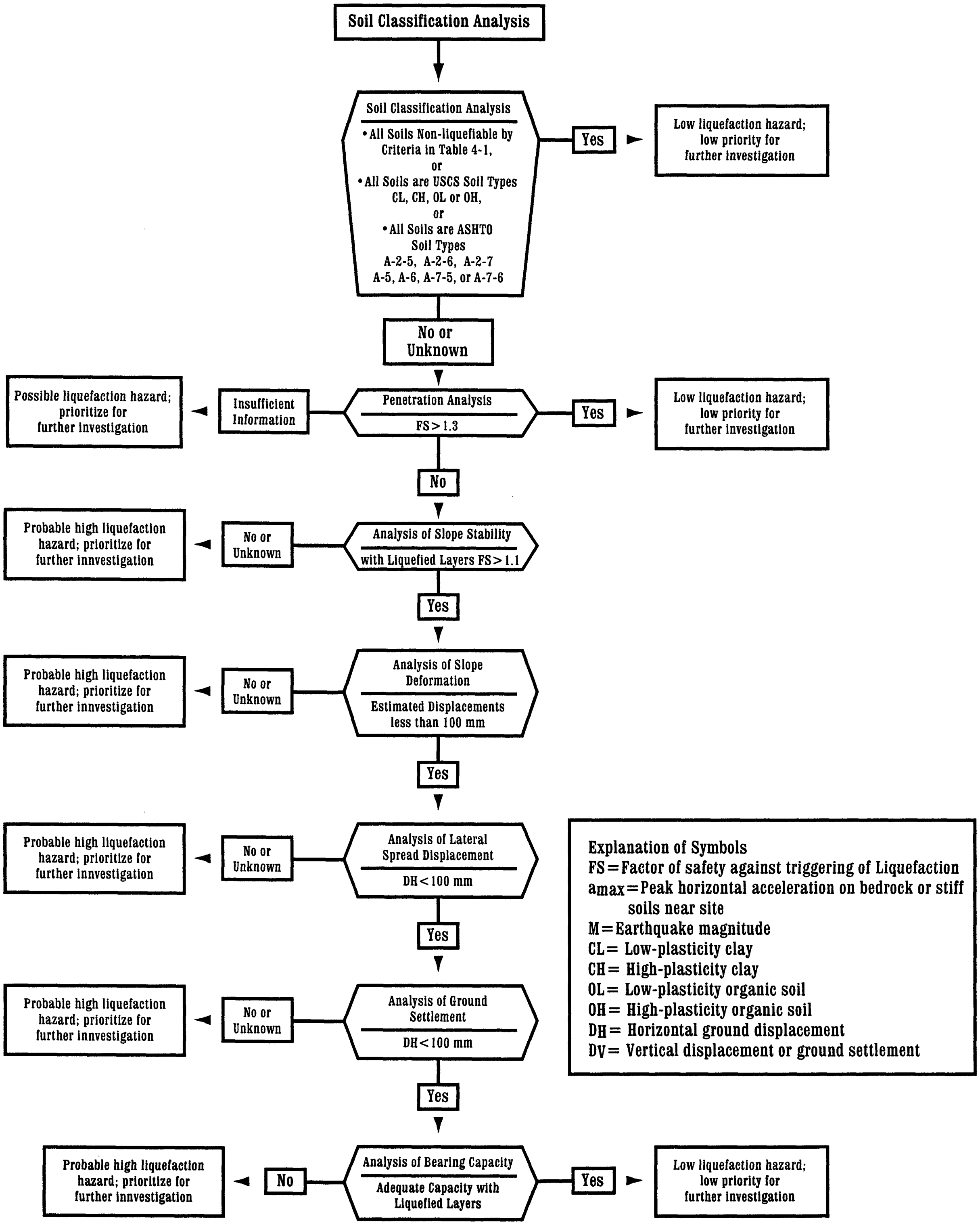


Figure 2-1. Flow Diagram Showing Steps and Criteria for Screening of Liquefaction Hazard for Highway Bridges--Part II

SECTION 3
REGIONAL EVALUATIONS OF LIQUEFACTION SUSCEPTIBILITY

3.1 Prior Evaluation of Liquefaction Hazard

If a prior evaluation of liquefaction hazard has been made for a site in question, or if a liquefaction hazard map has been compiled for the area or region in which the site is located, those results may be used as a screen for liquefaction hazard. Those evaluations and results should be reviewed and reevaluated using current criteria, such as those described in this guide. Where the verified evaluations indicate very low liquefaction hazard, those results may be used to classify the bridge site as low hazard and low priority for further investigation. Otherwise, the evaluation continues as noted below.

Youd (1991) and Power and Holzer (1996) have assembled lists of areas and regions in the U.S. for which liquefaction hazard maps have been compiled. Those listings include maps for areas in Alaska, California, New York, South Carolina, Utah, Washington, and parts of the central U.S that were affected by the New Madrid seismic zone. Each State Geologists should have copies or information on the availability of liquefaction hazard maps for his or her state.

3.1.1 Data Required

Information and data required for this level of screening include (1) reports containing previous assessments of liquefaction hazard for the bridge or highway segment in question or for nearby sites, (2) liquefaction hazard maps for the quadrangle or region in which the site or segment is located, and (3) reports of occurrences of liquefaction during past earthquakes at or near the bridge site. Information on the severity and distribution of past liquefaction occurrences may be found in earthquake reconnaissance reports, and compendiums of liquefaction occurrences. If prior reports, hazard maps, or information concerning past liquefaction are not available, the site is conservatively categorized as possibly liquefiable, and the evaluation proceeds to the next step as indicated in Figure 2-1.

3.1.2 Analysis and Classification

If a prior analysis of liquefaction hazard has been made for the site, the adequacy of that input data, the method of analysis, and the results from those studies should be reevaluated using the procedures and criteria listed in Section 4. If conservative input values were applied and if that review indicates very low hazard (safety factor of 1.3 or greater), then the site is classified as low hazard and low priority for further investigation.

The assessment may also be based on mapped liquefaction susceptibility. Liquefaction susceptibility maps have been compiled for several seismic areas in the U.S. Because there are variances in compilation techniques and definitions of hazard categories, the utility and meaning of hazard categories must be verified for each map. However, areas classified as "very low" liquefaction susceptibility or potential consistently define zones where the hazard to bridges is sufficiently low that liquefaction-induced damage is not expected. Bridges located in these areas may be conservatively classified as low hazard and low priority for further investigation. Bridge sites in areas zoned with higher hazard ratings should be further evaluated using the site-specific procedures listed in Section 4.

Prior occurrences of liquefaction at or near a bridge site indicate a possible high liquefaction hazard. Because liquefaction tends to recur at the same site during successive earthquakes, evidence of past liquefaction indicates a possibility of future liquefaction, providing site conditions, such as depth to ground water, have not changed (Youd, 1984). Reports from earthquake reconnaissance investigation are available for several parts of the U.S. Paleoseismic investigations have revealed additional sites where liquefaction occurred prehistorically. Where liquefaction has occurred previously, additional analysis is required to fully determine the current hazard, and the screening proceeds to the next step as noted in Figure 2-1. Lack of evidence of a prior occurrence of liquefaction does not provide adequate proof that a site is immune to liquefaction. Thus, where evidence is lacking, liquefaction is assessed as possible and the evaluation proceeds to the next step.

3.2 Geologic Analysis

As noted in the introduction, deposits susceptible to liquefaction are not randomly distributed in natural landscapes but occur within a rather narrow range of sedimentary environments. Saturated, loose, uncemented granular deposits are the most susceptible. Because density and cementation generally increase with age, liquefaction resistance also generally increases with age. Commonly used geologic criteria are listed in Table 3-1, which indicates that late Holocene deposits are most susceptible to liquefaction and that resistance generally increases with age through the Holocene and Pleistocene eras. Pre-Pleistocene sediments are generally immune to liquefaction and are rated as very low susceptibility.

The mode of deposition also has a major influence on liquefaction susceptibility. For example, sediments sorted into fine- and coarse-grained layers, such as by fluvial or wave actions, are generally more susceptible to liquefaction than unsorted sediments, such as glacial till. Processes that overconsolidate soils, such as glacial overriding, also reduce liquefaction susceptibility. Thus an evaluation of geologic units and depositional process can be used both as a screen for identification of low-hazard sites and as criteria for prioritizing potentially liquefiable sites for further investigation (Section 6).

3.2.1 Data Required

The information needed for geologic evaluation of liquefaction susceptibility is generally delineated on geologic maps or provided in geologic reports. The more detailed the information, particularly for Quaternary geologic or sedimentary units, the more useful the map or report. Geologic maps at a 1:24,000 scale (7-1/2 minute quadrangle scale) are widely available and may provide adequate

Table 3-1 Estimated Susceptibility of Sedimentary Deposits to Liquefaction during Strong Seismic Shaking (After Youd and Perkins, 1978)

Type of deposit	sediments in General distribution of cohesionless deposits	Likelihood that cohesionless sediments, when saturated, would be susceptible to liquefaction (by age of deposit)			
		< 500 yr	Holocene	Pleistocene	Pre-pleistocene
(1)	(2)	(3)	(4)	(5)	(6)
(a) Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	-	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
(b) Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach					
High wave energy	Widespread	Moderate	Low	Very low	Very low
Low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
(c) Artificial Fill					
Uncompacted fill	Variable	Very high	-	-	-
Compacted fill	Variable	Low	-	-	-

information if Quaternary geologic units have been adequately mapped. (Maps that designate all unconsolidated materials as Quaternary alluvium [Qal] are not very useful for liquefaction hazard evaluation.) For localities where adequate geologic maps are not available, local site maps may be prepared from a review of geologic literature, analysis of soil logs, or from site reconnaissance visits. An engineering geologist, or other geologist with background in Quaternary geologic units and processes, should be consulted for such local mapping projects.

If adequate geologic information is not available, and additional investigations are not feasible, the screening processes continues to the next step as noted in Figure 2-1.

3.2.2 Analysis and Classification

Geologic criteria developed by Youd and Perkins (1978), reproduced in Table 3-1, are appropriate and widely used for liquefaction hazard screening purposes. These criteria are based on qualitative analysis of geologic conditions at sites of past liquefaction. The criteria indicate that the younger, looser, and more segregated the deposit, the greater the susceptibility to liquefaction. Older (pre-Pleistocene), indurated, or cemented deposits are almost universally resistant to liquefaction. The geologic screening is performed by comparing geologic units mapped at a given bridge site with the criteria listed in Table 3-1 or with locally derived relationships, and a susceptibility rating is assigned to each unit. If all units are characterized by very low susceptibility, the site is classified as low liquefaction hazard and low priority for further investigation, and the evaluation of that site is complete. If the geologic criteria indicate a higher susceptibility rating, the site should be classified as possibly liquefiable, and the screening proceeds to the next step as indicated in Figure 2-1.

3.3 Evaluation of Seismic Hazard

A certain threshold of seismic energy must propagate through a site during seismic shaking to generate a liquefied condition. That threshold is a function of the density of the material, depth of sediment burial, and several other factors. Even the most susceptible natural materials requires a finite amount of seismic energy to generate a liquefied condition. Thus seismic energy from small local earthquakes or larger, distant events may be insufficient to generate liquefaction. Even after a liquefied condition is generated, additional seismic energy is usually required to induce damaging ground deformations or displacements. The seismic factors most commonly used by geotechnical engineers to characterize seismic energy propagating through a site are earthquake magnitude (M) and peak horizontal acceleration at ground surface (a_{max}). By assessing minimal values of earthquake magnitude and peak acceleration required to generate liquefaction and damaging ground deformations, threshold criteria can be developed for liquefaction hazard screening.

3.3.1 Data Required

Data required for screening on the basis of seismic input include estimates of maximum earthquake magnitude, M, for all possible earthquake sources pertinent to the site and corresponding estimates of peak horizontal acceleration, a_{max}, at the site to be evaluated. In regions where major earthquake sources are well defined, deterministic methods may be used to estimate M and a_{max}. Earthquake magnitudes are usually estimated from empirical correlations relating magnitude to fault length, fault slip, etc. Wells and Coppersmith (1994), for example provide useful correlations for estimating magnitude. a_{max} may be estimated from published correlations of peak ground motion parameters versus distance from the energy source for rock or stiff to moderately stiff sites (Boore et al., 1993; Kramer, 1996, p. 89). Different relationships may be required for different regions of the U.S. For example, peak ground motions attenuate more rapidly in the western U.S. than in the eastern U.S. The engineer should check to see if seismic hazard analyses have been made for other major structures in the area such as dams, power plants, and petroleum processing facilities, which might provide information useful to liquefaction hazard evaluations for bridges.

For regions with many or poorly defined seismic sources, use of probabilistic evaluations for M and a_{max} may be advantageous. National hazard maps showing contours of a_{max} with various probabilities of exceedance are published by the U.S. Geological Survey (USGS) and are available at the following website: http://geohazards.cr.usgs.gov/eq/. Several states have compiled probabilistic hazard maps specific to their needs. Where state seismic hazard maps have been compiled, those maps should be used. Because the state-of-knowledge of earthquake sources is rapidly changing, the latest widely accepted map should be used. For hazard screening, conservative values of M and a_{max} should be used. Values with 2% probability of exceedance in 50 years should be sufficiently conservative for most highway applications.

To assist the user, the 1996 USGS national map of peak acceleration with 2% probability of exceedance in 50 years is reproduced in Figure 3-1. Also, a map showing magnitudes of earthquakes with a recurrence interval of about 0.0004 per year (once in 2,500 years) is compiled in Figures 3-2 and 3-3 for the conterminous U.S. and areas of California, respectively. Magnitudes and recurrence information from which these maps were compiled were extracted from data tabulated by Hanson and Perkins (1995). These data form the bases for Figures 3-2 and 3-3, except that the author upgraded three zones (near Charleston, South Carolina; Boston, Massachusetts; and Klamath Falls, Oregon) to the next higher magnitude category to account for damaging historic earthquakes in those areas. Estimates of a_{max} and M may be taken directly from Figures 3-1, 3-2, and 3-3 for areas where better estimates are not available. The accuracy of these maps may be poor in some parts of the nation, and updated maps should be used if available.

3.3.2 Analysis and Classification

Analyses of sites that have or have not liquefied during past earthquakes indicate that for a given magnitude, a threshold peak acceleration must be exceeded to generate liquefaction. Conservative estimates of these thresholds, even for very susceptible sediments, are listed in Table 3-2. Based on statistical analyses by Liao et al. (1988) and Youd and Noble (1997), these threshold conditions generate about a 5% probability of liquefaction in very susceptible sediments. If estimates of maximum earthquake magnitudes and peak acceleration do not exceed these thresholds, the site may be classified as low liquefaction hazard and low priority for further investigation. If these thresholds are exceeded, the screening proceeds to the next step (Figure 2-1).

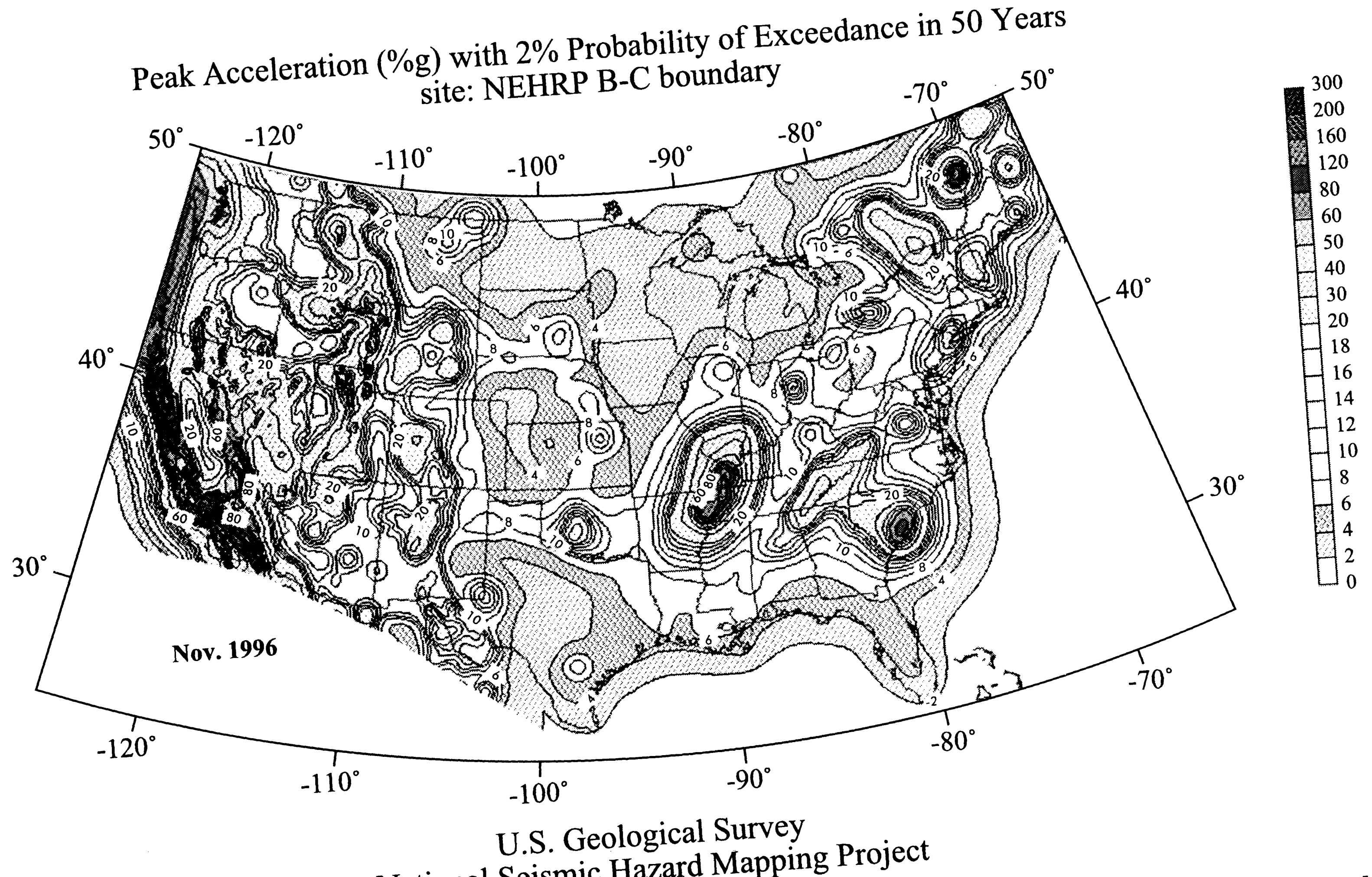


Figure 3-1. 1996 U.S. Geological Survey Map Showing Peak Horizontal Ground Accelerations, a_{max}, on Bedrock or Stiff-Soil Profiles with 2% Probability of Exceedance in 50 Years (http://geohazards.cr.usgs.gov/eq/)

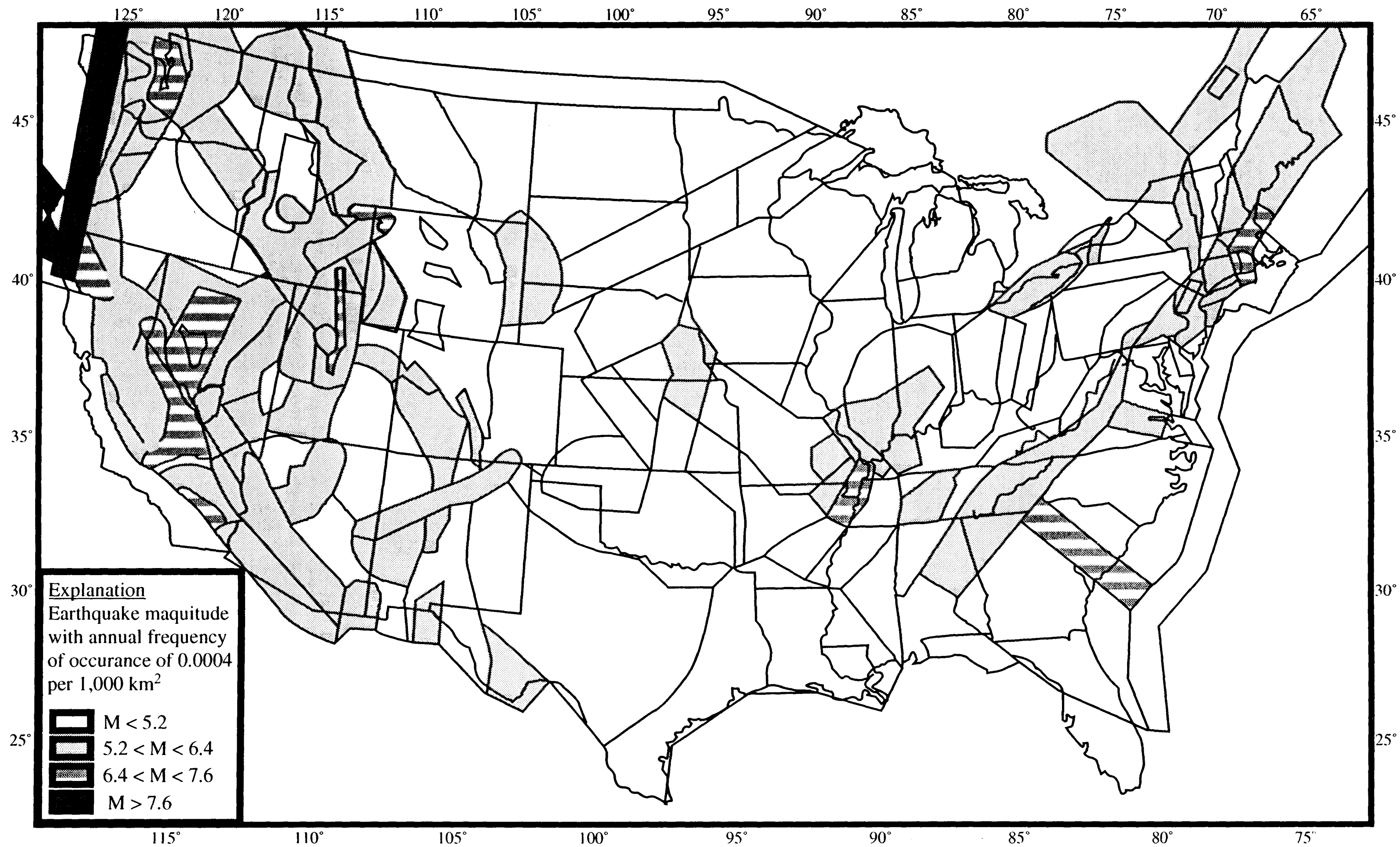


Figure 3-2. Map of Conterminous U.S. Showing Seismic Source Zones and Earthquake Magnitudes with about a 0.0004 Annual Rate of Occurrence (2,500-Year Recurrence Interval) (Data from Hanson and Perkins, 1995)

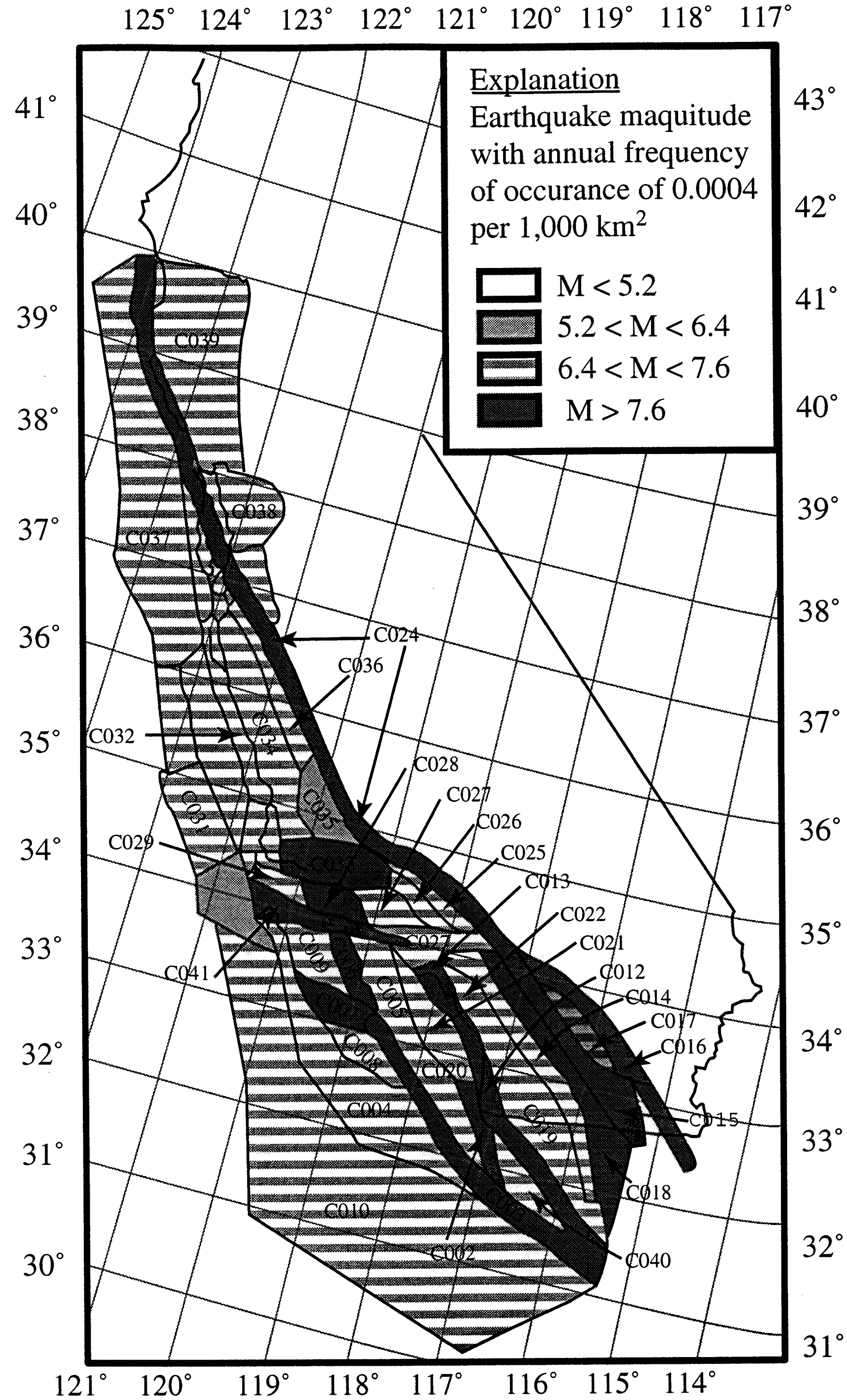


Figure 3-3. Map of Western California Showing Seismic Source Zones and Earthquake Magnitude with about a 0.0004 Annual Rate of Occurrence (2,500-Year Recurrence Interval) (Data from Hanson and Perkins, 1995)

Table 3-2 Minimum Earthquake Magnitudes and Peak Horizontal Ground Accelerations, With Allowance for Local Site Amplification, that Are Capable of Generating Liquefaction in Very Susceptible Natural Deposits

Earthquake Magnitude[1] M_w	**Liquefaction Hazard for Bridge Sites**	
	Soil Profiles Types[2] I and II (stiff sites)	Soil Profile Types[2] III and IV (soft sites)
$M < 5.2$	Very low hazard for $a_{max} <0.4g$	Very low hazard for $a_{max} <0.1g$
$5.2 < M < 6.4$	Very low hazard for $a_{max} <0.1g$	Very low hazard for $a_{max} <0.05g$
$6.4 < M < 7.6$	Very low hazard for $a_{max} <0.05g$	Very low hazard for $a_{max} <0.025g$
$M > 7.6$	Very low hazard for $a_{max} <0.025g$	Very low hazard for $a_{max} <0.025g$

[1]Magnitudes are at the upper end of the magnitude intervals given by Hanson and Perkins (1995).
[2]Soil Profile Types, as defined in Standard Specifications For Highway Bridges (AASHTO, 1992).

For deterministic estimates of M and a_{max}, the criteria in Table 3-2 may be applied directly for initial screening. For probabilistic analyses, an a_{max} with 2% probability of exceedance in 50 years (approximate 2,500 year recurrence interval) should be used, along with the maximum size earthquake with an annual rate of occurrence of about 0.0004 (once in 2,500 years). These values may be taken from hazard maps, such as those given in Figures 3-1, 3-2, and 3-3, or more preferably from updated versions of these maps as they become available. In either instance, this procedure is conservative because the most likely sources of peak accelerations plotted in Figure 3-1 are earthquakes with magnitudes substantially smaller than earthquakes with a 0.0004 probability of occurrence.

3.4 Water Table Analysis

Liquefaction occurs only in saturated or very nearly saturated materials. (A very few instances of liquefaction of very loose, dry soils have been reported, but those instances are rare, have not occurred in the U.S., and can be neglected for liquefaction hazard evaluation of U.S. bridge sites.) Only saturated sediments or sediments capable of becoming saturated with ground water should be considered as susceptible to liquefaction. Sediments beneath a free groundwater table, permanent or perched, are generally saturated. Because liquefaction resistance increases with overburden pressure and age of sediment, both of which generally increase with depth, liquefaction resistance usually increases markedly with water table depth. Thus the deeper the water table, the greater the resistance of sediments to liquefaction.

Because liquefaction resistance generally increases with depth, most past occurrences of liquefaction in natural sediments have occurred at shallow depths. Geotechnical investigations at sites of past

Table 3-3 Relative Liquefaction Susceptibility of Natural Sediments as a Function of Groundwater Table Depth

Groundwater Table Depth	Relative Liquefaction Susceptibility
< 3 m	Very High
3 m to 6 m	High
6 m to 10 m	Moderate
10 m to 15 m	Low
> 15 m	Very Low

liquefaction indicate that most episodes of liquefaction have developed in areas with a water table shallower than 3 m; some episodes have occurred in areas with water tables as deep as 10 m; a few episodes have developed beneath water tables deeper than 15 m. (These relationships are not valid for thick man-made fills, such as embankment dams, dikes, or causeways.) Based on these observations, the watertable depth relationships listed in Table 3-3 were developed for screening liquefaction hazard at bridge sites.

3.4.1 Data Required

The only field measurement required for this assessment is depth to the unconfined groundwater table, either permanent or perched. Because groundwater levels at many sites fluctuate seasonally or with longer-term climatic conditions, the highest average or likely groundwater level under adverse climatic conditions should be estimated. A reasonably conservative estimate might be the highest level expected over a typical 20-year period. Where water levels sporadically rise for very short periods of time, such as may occur infrequently beneath usually dry creek beds or arroyos in desert regions, these short-term peaks may be neglected in evaluating longer-term water table depths.

Where groundwater levels have been monitored over several years, which is unusual at bridge sites, the highest recorded levels should be used for liquefaction hazard evaluation. Where levels were measured only at the time of drilling of exploratory holes for foundation investigations, which is the more usual procedure, some upward adjustment to the measured groundwater level may be needed to provide a conservative estimate of expectable high groundwater levels. Factors that should be considered in making that adjustment include time of year and wetness of the season in which the water level was measured, and typical fluctuations in groundwater level that have been measured at sites with similar hydrologic and topographic settings. Consultation with a groundwater hydrologist familiar with the area may be required to develop reasonable estimates of expectable high groundwater levels. For bridge sites with several meters of topographic relief, such as bridge crossings over incised streams, the shallowest depth to free groundwater beneath the steam bed should be used in this screening analysis. Thus sites with bridges crossing permanently flowing streams would always be classed as possibly liquefiable by the watertable criteria.

Where foundation investigation reports are not available, or where groundwater levels were not recorded on borehole logs, less certain sources of information may be used. Groundwater measurements at nearby sites may provide guidance on areal groundwater levels. Maps showing groundwatertable elevations may also be available for some regions. Judgment is often required when using these non site-specific measures to weigh the influence of confined groundwater (artesian conditions) and the possibility of perched groundwater, which may lead to saturation at levels above the general groundwater level. Where watertable maps are not available, a groundwater hydrologist or other authority familiar with groundwater conditions in the area may provide useful estimates of groundwater depths. These estimates of groundwater depth may be used in screening analysis, provided adequately conservative assumptions are applied.

3.4.2 Analysis and Classification

Watertable depth criteria for use with this screening guide are listed in Table 3-3. These criteria indicate that damaging episodes of liquefaction are unlikely to developing at depths greater than 15 m. Thus if the long-term watertable persistently remains deeper than 15 m, the site may be classified as low likelihood of liquefaction and low priority for further investigation. For groundwater levels above 15 m, the evaluation continues to the next step as indicated in Figure 2-1.

SECTION 4
SITE-SPECIFIC EVALUATIONS OF LIQUEFACTION RESISTANCE OF SANDS AND STRENGTH-LOSS IN SENSITIVE FINE-GRAINED SOILS

4.1 Screening for Extra Sensitive Clays

A phenomenon closely related to liquefaction is strength loss in extra-sensitive clays (sensitivities greater than 4). Liquefaction develops because of a tendency for uncemented granular soils to compact as a consequence of earthquake-induced cyclic shear deformation. Thus liquefaction occurs only in granular soils with sufficiently low clay content and plasticity that clay bonding between particles does not inhibit compaction during seismic shaking (Section 2.6). Strength loss in extra-sensitive clays, is caused by the collapse of metastable flocculated, or "cardhouse-type," structures in fine-grained soils. These soils contain large fractions of colloidal- or clay-size particles. Usually the metastable state is caused by the leaching of salts from interstitial water within fine-grained soils. The most susceptible soils are those that were originally deposited in a marine or saline environment and later permeated with fresh water. Extra-sensitive clays are rather rare in nature and are most commonly found in post-glacial clays deposited in saline seas or lakes (Mitchell, 1993).

During a few earthquakes, sensitive soils have softened, lost strength, and allowed large ground displacements to occur. Notably, the 1964 Alaska earthquake (M_w = 9.2) triggered five large landslides within urban areas of Anchorage, causing severe damage to commercial and residential facilities. Although liquefaction of sand lenses may have had some influence in generating these landslides, the failure zones formed primarily within sensitive facies of the Bootlegger Cove formation. Measured sensitivities from the most sensitive parts of the Bootlegger Cove formation ranged from 6 to 26, liquid limits ranged from 27% to 39%, moisture contents ranged from 27% to 37%, and clay contents (percent finer than 2μ) ranged from 18% to 58%. Nearly all highly sensitive soils classify as low-plasticity clay (CL) (Mitchell et al., 1973).

Several simple criteria may be used as screens for sensitive clays. These criteria include classifications as low-plasticity "soft soils" with sensitivities greater than 4 and liquidity indexes greater than 0.6. Such conditions are nearly always associated with natural moisture content greater than 0.9 times the liquid limit, a criterion also used for evaluating liquefaction hazard in fine-grained soils. Soft sensitive soils also have low penetration resistances, characterized by $(N_1)_{60}$ less than 5 or corrected and normalized CPT tip resistances, q_{c1N}, less than 1 MPa. Based on these criteria, only UCS classifications of CL and ML and AASHTO classifications of A-4, A-2-4, A-6, and A-2-6 are capable of developing high sensitivity. Because sensitive clays are relatively rare in the U.S., additional screening criteria include reported deposits of sensitive clays in the area or past or present depositional environments capable of producing sensitive soils, such as saline lakes or seas. Extra-sensitive soil deposits have been reported in only a few areas of the U.S., primarily in Alaska and along the St. Lawrence River valley. Sensitive soils also may have formed in lacustrine sediments laid down in ancient saline lakes of the Great Basin region or in esturine sediments that commonly lie along coastal rivers or bays. Residual soils developed from weathered volcanic ash may also contain sensitive layers.

4.1.1 Data Required

The primary information required for screening extra-sensitive clays are standard soil properties used for classifying and defining soil state. These properties include penetration resistance, Atterberg limits, clay content, and natural moisture content. These data are commonly listed on foundation investigation borehole logs. Classifications made by either the AASHTO, or Unified classification systems may also be used for hazard screening. In addition, reports of extra-sensitive deposits or environments capable of producing sensitive soils in an area may be used as screening criteria.

4.1.2 Analysis and Classification

Based on the criteria noted above, sensitive soils are restricted to fine-grained sediments with a liquid limit less than 40%; a moisture content greater than 0.9 times the liquid limit (liquidity index greater than 0.6); a corrected standard resistance, $(N_1)_{60}$, less than 5; or a corrected and normalized cone penetration resistance, q_{c1N}, less than 1 MPa. Fine-grained soils with property values that exceed any one of these limits may be safely classed as nonsensitive and not vulnerable to strength loss during seismic shaking. If some soil layers meet the criteria listed above, or if some information is unknown, the analysis for extra-sensitive clays should continue through the following steps.

Only UCS soil types CL or ML and AASHTO soil types A-4, A-2-4, A-6, and A-2-6 meet the above criteria. If the site in question is devoid of these soil types, fine-grained soils at the site are not susceptible to strength loss, and the screening continues on to the next step.

A final screen that should be applied is the known presence of sensitive clays in the area or depositional environments capable of producing sensitive clays. These areas are relatively rare in the U.S., lying primarily in coastal areas of Alaska, such as Anchorage, along the St. Lawrence River, in esturine soils along both coasts, and near saline lakes in the Great Basin and other arid areas. If sensitive soils have not been identified in localities near the bridge site, and the site is not in one of the areas noted above, sensitive soils are unlikely. In such instances, the site may be classed as unlikely to contain sensitive soils, and the screening proceeds to the next step. If the site is underlain by one or more layers of soil meeting the criteria noted above, and the site is in an area where sensitive soils may have developed, then the site should be classed as possibly underlain by extra-sensitive soils and given high priority for additional investigation (Figure 2-1).

4.2 Soil Classification Evaluation

Liquefaction is generally restricted to coarse-grained soils (silts, sands, and gravels) that are sufficiently loose and uncemented to induce a tendency to compact during seismic shaking. In undrained or poorly drained soils, this tendency commonly generates increased pore pressures and a liquefied condition. Clay bonding between particles generally inhibits seismic compaction of fine-grained soils and cohesive coarse-grained soils, preventing the occurrence of liquefaction. Thus these types of soils are generally resistant to liquefaction. Based on observed reported behavior of cohesive soils in areas of strong ground shaking, primarily from China, Seed and Idriss (1982)

Table 4-1 Criteria for Assessing Liquefiability of Fine-Grained Soils (modified from Seed and Idriss, 1982)

Criteria Required for Liquefaction of Fine Grained Soils (All three criteria must be met for soil to be liquefiable)
■ Clay Fraction (Percent Finer Than 0.005 mm) < 15% ■ Liquid Limit (LL) < 35% ■ Moisture Content (MC) > 0.9 LL

developed the criteria listed in Table 4-1. These criteria, commonly referred to as " the Chinese criteria," are widely used in the geotechnical profession and provide generally conservative predictions of liquefaction behavior.

Koester (1992) reexamined these criteria and the testing procedures used in each country and suggested several modifications, including reducing the value of the liquid limit criterion from 35% to 31%. To be conservative and consistent with general geotechnical engineering practice, the criteria listed in Table 4-1 are recommended for screening liquefaction hazard based on soil type or classification.

Soils at bridge sites are commonly classified by either the Unified Classification System (UCS) or the AASHTO classification system. These classifications may be used for liquefaction hazard screening purposes. UCS divides soils into coarse-grained and fine-grained materials based on the percentage of fines. Soils with more than 50% particles, by weight, passing a No. 200 sieve are classed as finegrained. Using the plasticity chart plotted in Figure 4-1A, fine-grained soils are further classified into clays (C) or silts (M) with high (H) or low (L) plasticity. Organic soils (O) are also divided into high- and low-plasticity types. Coarse-grained soils with more than 12% fines are also classified using the plasticity chart in Figure 4-1A. Based on whether the plasticity information plots above or below the A-line, respectively, coarse-grained soils with more than 12% fines are classed as clayey sands (SC), clayey gravels (GC), silty sands (SM), or silty gravels (GM).

The following logical assumption is required for liquefaction hazard screening: all soils containing a character "C" in the two-letter Unified Classification designation (indicative of clay) contain more than 15% clay and thus are not liquefiable by the criteria in Table 4-1. Theoretically, a soil with 12% fines could be classified as SC or GC and still meet the criteria in Table 4-1 for a liquefiable soil. Such a soil, however, is virtually impossible in nature because the fraction of the soil finer than 0.42 mm (No. 40 sieve) would also have to have a liquid limit greater than 10 and a plasticity index greater than 7. That level of plasticity generally requires a fines content much greater than 12% and a clay content greater than 15%. Thus the assumption that soils with a "C" descriptor in the two-letter Unified Classification code are nonliquefiable is valid for practical purposes. Based on this assumption, all layers of soil designated as CH (high-plasticity clay), CL (low-plasticity clay), SC

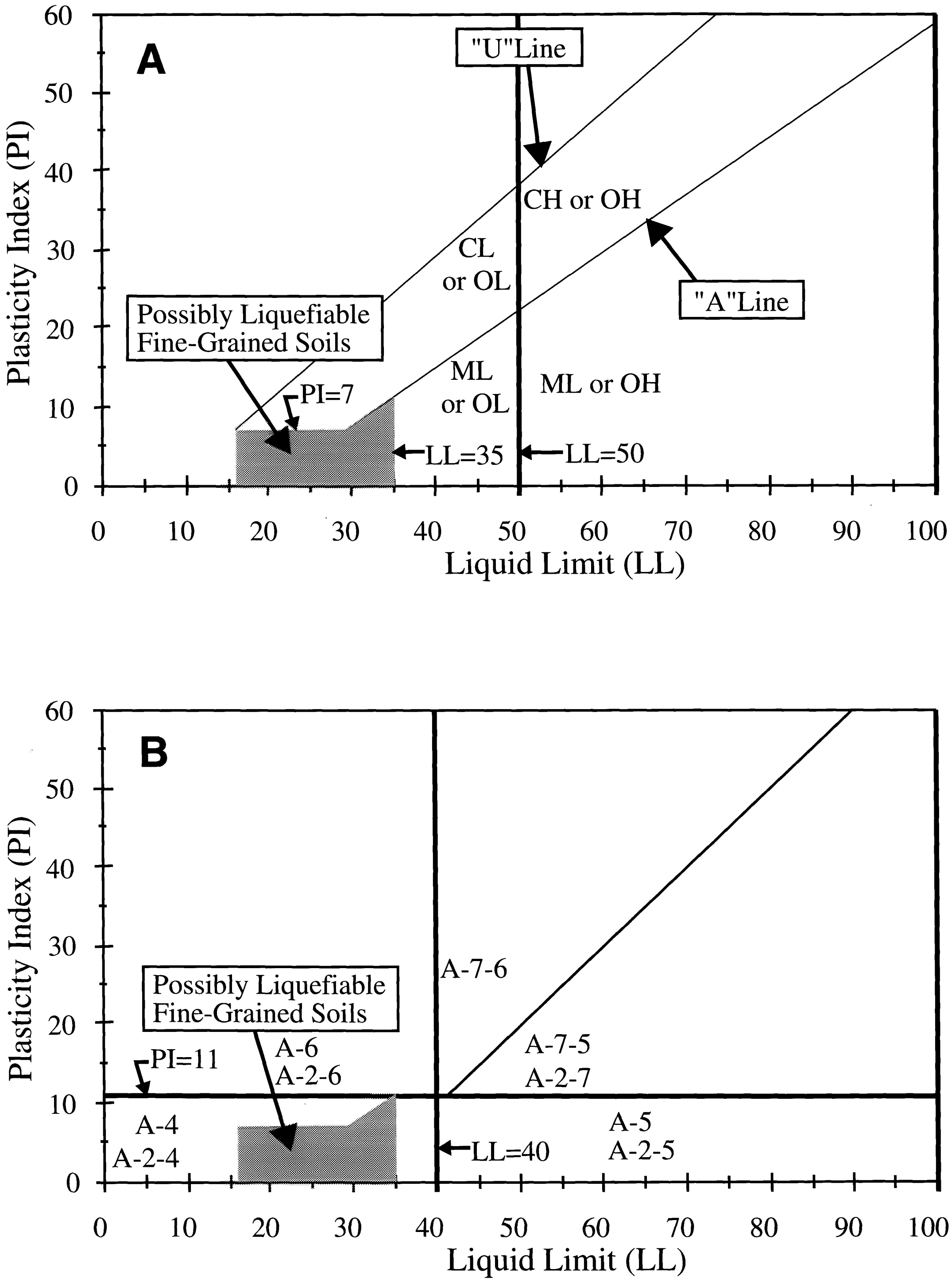


Figure 4-1. Chart Showing (A) Unified Soil Classification System (UCS) and (B) AASHTO Classification System for Fine-Grained Soils with Stippled Region of Chart Indicative of Possibly Liquefiable Soils

(clayey sand), or GC (clayey gravel) are screened as nonliquefiable. To be conservative, soils classed with a dual designation--CL-ML, SM-SC, or GM-GC (plasticity indices between 5 and 7)--could possibly have clay contents less than 15% and are classed herein as potentially liquefiable. With the assumption that all soils with a "C" in the two-letter descriptor are nonliquefiable, potentially liquefiable soils are shown by the stippled pattern on Figure 4-1A. These soils lie below the A-line, have a liquid limit less than 35, and are classed as nonclayey and low plasticity (liquid limit less than 50).

Figure 4-1B shows the region of the plasticity chart identified as possibly liquefiable in Figure 4-1A transferred onto the AASHTO classification chart. Based on this chart, only fine-grained soil types A-4 and A-2-4 are liquefiable, whereas soil types A-2-5, A-2-6, A-2-7, A-5, A-6, A-7-5, and A-7-6 are immune to liquefaction. Because possibly liquefiable soils occupy only part of the A-4 and A-2-4 regions, further analysis may identify some A-4 and A-2-4 soils as nonliquefiable.

The third criterion in Table 4-1 for assessing liquefiability of fine-grained soils is that the moisture content for liquefiable soils must be greater than 0.9 times the liquid limit (LL). Fine-grained soils with natural moisture contents less than 0.9 LL are generally moderately to highly over-consolidated and not susceptible to liquefaction.

4.2.1 Data Requirements

The primary data required for soil classification screening are Atterberg limits, clay content, and natural moisture content. Much of this data may be listed on borehole logs. Where classifications have been made by either the AASHTO or Unified Classification systems, those classifications may also be used for hazard screening.

4.2.2 Analysis and Classification

For sites where laboratory tests have been conducted to define the properties listed in Table 4-1, those criteria may be applied directly to define soil layers that are immune to liquefaction. If all soil layers below the watertable are classed as nonliquefiable by these criteria, then the site is classed as low liquefaction hazard and low priority for further investigation. If potentially liquefiable soil layers are present, the screening continues to the next step as indicated in Figure 2-1.

Where reported Atterberg limit or moisture content data are inadequate to apply the criteria in Table 4-1, but careful field soil classifications have been made, the soil classification criteria delineated in Figure 2-1 may be applied. Additionally, field classifications generally include descriptions of stiffness as well as estimates of soil type. Stiff to very-stiff fine-grained soils generally are over consolidated and have moisture contents less than 0.9 times the liquid limit. Such soils may also be classed as nonliquefiable. Although there may be some uncertainty in the correctness of field classifications, the information is generally adequate if conservatively interpreted for liquefaction hazard screening. Again, if all soil layers below the watertable are assessed as nonliquefiable by these criteria, the site may be classified as low liquefaction hazard and low priority for further investigation. If possibly liquefiable layers lie beneath the watertable, or if the available information is inadequate for screening based on soil classification, the evaluation continues to the next step as indicated in Figure 2-1.

4.3 Penetration Analysis

4.3.1 Simplified Procedure

Over the past 25 years, a procedure termed the "simplified procedure" has evolved for evaluating liquefaction resistance of soils. This procedure has become the standard of practice in North America and throughout much of the world. Following disastrous earthquakes in Alaska and Niigata, Japan, in 1964, Seed and Idriss (1971) developed and published the original "simplified procedure." The procedure has been corrected and augmented periodically since that time with landmark papers by Seed (1979), Seed and Idriss (1982), Seed et al. (1985) and NRC (1985). More recently, the National Center for Earthquake Engineering Research, NCEER, sponsored workshop to consider additional updates and revisions to the simplified procedure (Youd and Idriss, eds., 1997). The following text incorporates consensus recommendations adopted by that workshop.

The calculation, or estimation, of two primary seismic variables are required to evaluate liquefaction resistance. These variables are the demand placed on a soil layer, expressed in terms of cyclic stress ratio (CSR), and the capacity of the soil to resist liquefaction, expressed in terms of cyclic resistance ratio (CRR), hereafter termed as liquefaction resistance or liquefaction resistance ratio. The factor of safety against liquefaction in terms of these two variables is written as:

$$FS = CRR/CSR \tag{4.1}$$

Seed and Idriss (1971) formulated the following equation for calculation of CSR:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \tag{4.2}$$

where a_{max} is the peak horizontal acceleration generated by the earthquake; g is the acceleration of gravity; σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively; and r_d is a stress reduction coefficient.

Curves showing the range and average values of r_d, as developed by Seed and Idriss (1971), are plotted in Figure 4-2. For noncritical projects such as hazard screening, the following equations may be used to estimate average values of r_d for use in Equation 4-2:

$$r_d = 1.0 - 0.00765\ z \qquad \text{for } z \leq 9.2 \text{ m} \tag{4.3a}$$
$$r_d = 1.174 - 0.0267\ z \qquad \text{for } 9.2 \text{ m} < z \leq 23 \text{ m} \tag{4.3b}$$
$$r_d = 0.744 - 0.008\ z \qquad \text{for } 23 \text{ m} < z \leq 30 \text{ m} \tag{4.3c}$$
$$r_d = 0.50 \qquad \text{for } z > 30 \text{ m} \tag{4.3d}$$

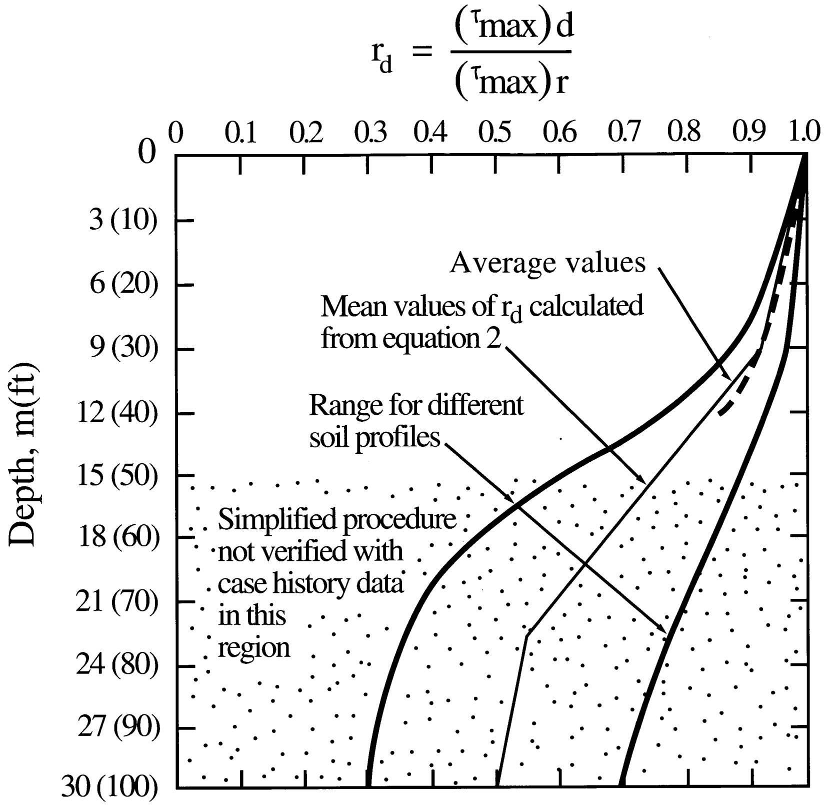


FIGURE 4-2 r_d Versus Depth Curves Developed by Seed and Idriss (1971), with Added Mean Value Lines from Equation 4-3 (after Youd and Idriss, 1997)

where z is depth below ground surface in meters. Average values of r_d estimated from these equations are plotted on Figure 4-2.

Several procedures have been applied to determine CRR. One plausible method is to extract undisturbed soil specimens from field sites and test those specimens using cyclically loaded laboratory tests to model seismic loading conditions. Unfortunately, specimens of granular soils retrieved with standard drilling and sampling techniques are generally too disturbed to yield meaningful laboratory test results. Only through specialized sampling techniques, such as ground freezing, can sufficiently undisturbed specimens be retrieved. The cost of such procedures is generally prohibitive for all but the more critical projects. To avoid the difficulties associated with sampling and testing, field tests have become the state-of-the-practice for routine investigations.

Several field tests have gained common usage for evaluation of liquefaction resistance, including the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements (V_s), and the Becker penetration test (BPT). Basic advantages and disadvantages of

Table 4-2 Comparison of Advantages and Disadvantages of Various Field Tests Used to Assess Liquefaction Resistance (modified from Youd and Idriss, 1997)

Feature	Test Type			
	CPT	SPT	V_s	BPT
Number of test measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Drained, large strain	Partially drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Very good	Poor to good	Good	Poor
Detection of variability of soil deposits	Very good	Good	Fair	Fair
Soil types in which test is recommended	Non-gravel	Non-gravel	All	Primarily gravel
Test provides sample of soil	No	Yes	No	No
Test measures index or engineering property	Index	Index	Engineering property	Index

each test are listed in Table 4-2. In past practice, standard penetration tests have been the most commonly used test in foundation investigations for bridges and other structures. Criteria are presented below for evaluating liquefaction resistance using this test. The CPT is becoming widely used for foundation engineering investigations; the criteria to use CPT data are also presented below. Shear-wave velocities and Becker penetrations resistances rarely have been measured in past foundation engineering practice. Hence, the latter tests are not discussed further herein, but relevant procedures and criteria are given by Youd and Idriss (1997).

4.3.2 Standard Penetration (SPT) Resistance

Criteria for evaluating liquefaction resistance based on standard penetration test (SPT) blow counts have been rather robust over the years. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ plot reproduced in Figure 4-3. That figure shows calculated CSR values plotted against $(N_1)_{60}$ data from sites where liquefaction effects were or were not observed following past earthquakes.

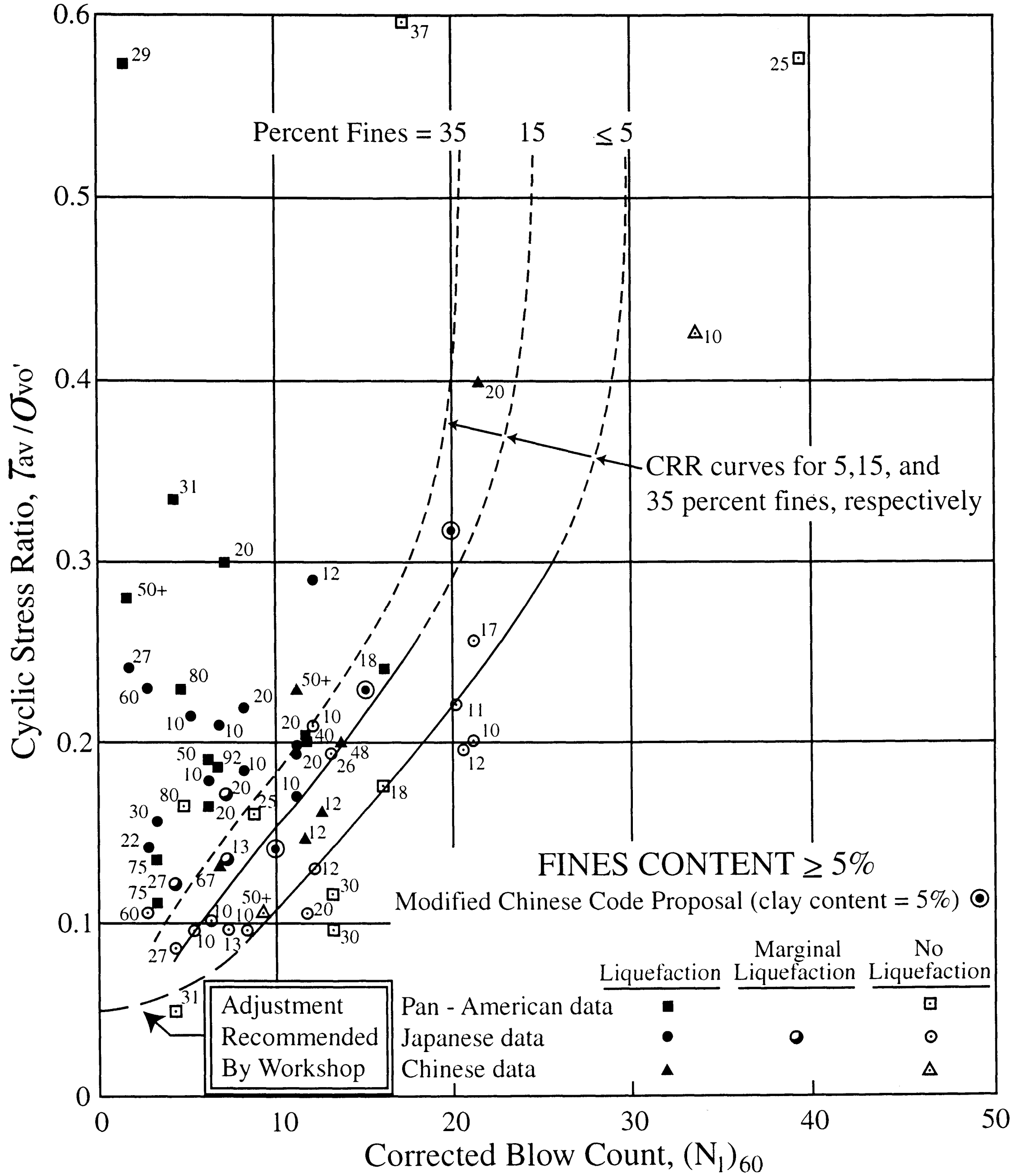


FIGURE 4-3 Simplified Base Curve Recommended for Calculating CRR from SPT Data along with Empirical Liquefaction Data (after Youd and Idriss, 1997)

Conservatively drawn CRR curves separate data indicative of liquefaction from data indicative of nonliquefaction for various fines contents. The CRR curve for magnitude 7.5 earthquakes and for fines contents less than 5% is the basic penetration criterion for the simplified procedure and is referred to hereafter as the "simplified base curve."

The simplified base curve can be approximated by the following equation:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \tag{4-4}$$

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes, $x = (N_1)_{60cs}$ (standard penetration resistance corrected to a clean-sand equivalent value), a=0.048, b=-0.1248, c=-0.004721, d=0.009578, e=0.0006136, f=-0.0003285, g=-1.673E-05, and h=3.714E-06. This equation is valid for $(N_1)_{60}$ less than 30 and may be used in spreadsheets and other analytical techniques to approximate the simplified base curve for engineering calculations.

4.3.2.1 Data Required

Data required to apply the simplified procedure include magnitude (M) and peak acceleration (a_{max}), soil stratigraphy and depth to the groundwater table, fines content or soil type, unit weight, and a measure of penetration resistance provided by SPT soundings. Estimation of these factors and properties for liquefaction screening purposes are described below.

Seismologists commonly calculate earthquake magnitude using any one of five different scales: (1) local or Richter magnitude, M_L; (2) surface-wave magnitude, M_s; (3) short-period body-wave magnitude, m_b; (4) long-period body-wave magnitude, m_B; and (5) moment magnitude, M_w. Moment magnitude is most commonly used for engineering applications and is the scale preferred for calculating liquefaction resistance. As shown in Figure 4-4, magnitudes from other scales may be substituted for M_w within the following limits: $M_L < 6$, $m_B < 7.5$ and $6 < M_s < 8$. m_b, a scale commonly used in the eastern U.S., may be used for magnitudes between 5 and 6, provided such magnitudes are corrected to M_w using the curves plotted in Figure 4-4 (Idriss, 1985).

The maximum acceleration, a_{max}, commonly used in liquefaction analysis is the peak horizontal acceleration that would occur at ground surface in the absence of excess pore-pressure or liquefaction. Thus the a_{max} used in Equation 4.1 is corrected for local site response, but not for excess pore-water pressures that might develop. For highway engineering purposes, a_{max} should be consistent with the maximum acceleration used for structural design of the bridge in question or other nearby bridges. Methods commonly used to estimate a_{max} include the following:

(1) The use of peak acceleration attenuation curves compatible with soil conditions at the bridge site are preferred for estimating a_{max}. Where soil conditions are not compatible with site conditions

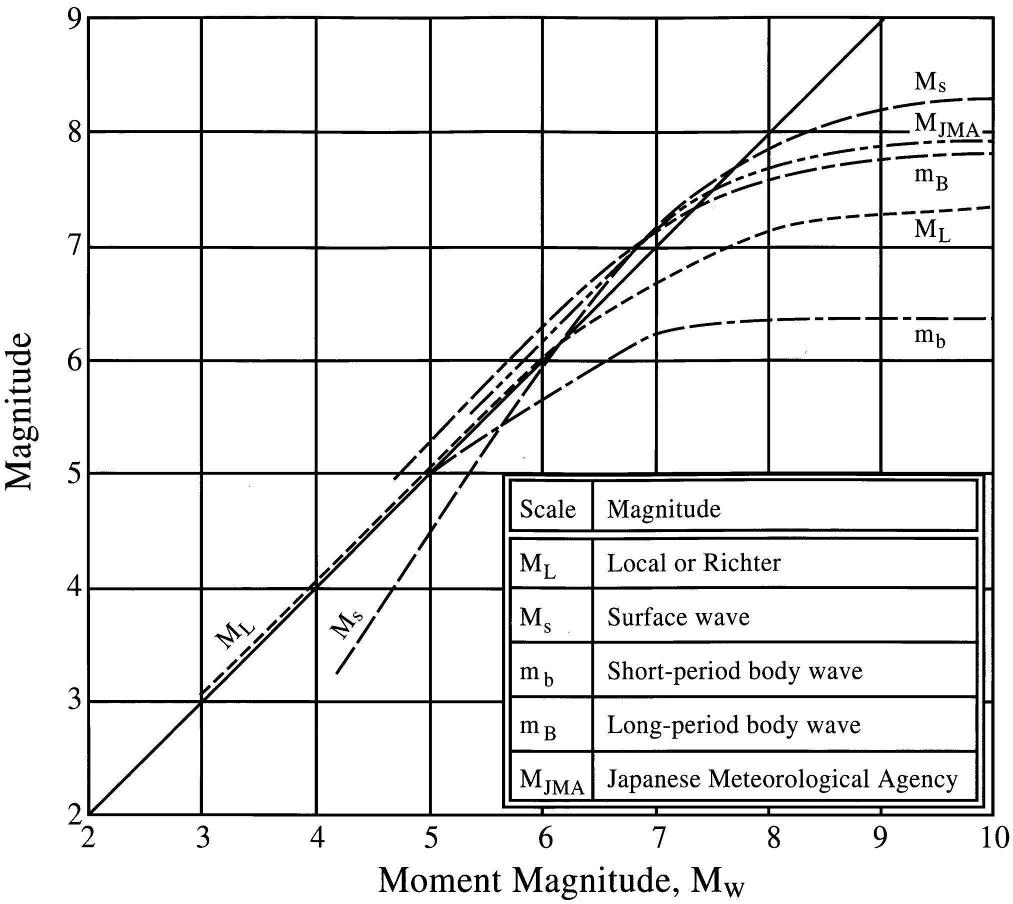


FIGURE 4-4 Relationship between Moment, M_w, and Other Magnitude Scales (After Heaton et al., 1982)

specified for various attenuation correlations, a_{max} may be estimated from a site-specific response analysis. Computer programs such as SHAKE and DESRA may be used in these analyses. A least desirable method for estimating peak ground acceleration at a site is through application of amplification ratios such as those developed by Idriss (1990; 1991). These ratios may be used to multiply bedrock outcrop motions to estimate surficial motions at soil sites. Because amplification ratios are magnitude and frequency dependent, caution and engineering judgment are required when applying these relationships.

(2) Peak accelerations may also be estimated from regional or national probabilistic maps of a_{max}. Again, if site conditions are different from those specified on the map, a_{max} should be corrected for local site conditions. A conservative value of a_{max} should be selected, typically a value with 2% probability of exceedance in 50 years. If probabilistic maps are used, the magnitude of the causative earthquake may not be obvious, and additional assumptions or investigation may be necessary. For example, the maximum magnitudes mapped on Figure 3-2 or 3-3 may be used, but those magnitudes

would likely be very conservative because the peak accelerations mapped on Figure 3-1 are usually influenced more by smaller and more frequent local earthquakes than by the larger 2,500-year events. An engineering seismologist might be consulted to assist to define a_{max} for screening applications.

The depth of the watertable is required to calculate σ'_{vo} and σ_{vo} and hence CSR. The watertable depth is usually recorded on borehole logs. As noted in Section 2.4, seasonally high groundwater levels should be used for screening applications. Thus groundwater levels recorded on borehole logs may have to be adjusted upward to account for seasonal or longer-term water-level variations.

Unit weights of the various soil layers encountered beneath a site are also used to calculate σ_{vo} and σ'_{vo}. Unit weights are seldomly recorded on borehole logs; thus this property is commonly estimated from typical values for the various soil types. Fortunately, CSR is not very sensitive to unit weight, and small errors in estimating this property are not very critical. Where no other guidance is available, the following unit weights, γ, may be conservatively assumed. For typical alluvial, lacustrine, deltaic, or esturine sediments, dry unit weights are usually 17 kN/m^3 (110 lb/ft^3) or less, moist unit weights are usually 19 kN/m^3 (120 lb/ft^3) or less, and saturated unit weights are usually 21 kN/m^3 (130 lb/ft^3) or less. Use of unit weights from the higher end of the range leads to larger estimates of CSR, which is conservative.

Fines contents are typically recorded on borehole logs or in soil test data in foundation reports. Fines contents are used to convert measured blow counts to clean-sand equivalent blow counts. Where fines contents are not given, low estimates from soil descriptions or assuming clean sand will provide conservative results.

Measured penetration resistances, N_m, are the primary data required for evaluating liquefaction resistance with SPT criteria. Procedures for conducting SPT are specified in ASTM D 1586. Additional information for conducting and interpreting SPT results for liquefaction resistance analysis are noted below. A variety of test equipment and procedures has been used in routine engineering practice--some of which do not meet ASTM standards. Because of these variances, a number of corrections have been developed to convert measured standard penetration resistances to approximate standard values.

4.3.2.1.1 Calculation of Corrected Penetration Resistance, $(N_1)_{60}$

Several corrections are required to convert measured standard penetration resistance, N_m, to an equivalent clean-sand standard penetration resistance, $(N_1)_{60cs}$, for use in calculating CRR from Equation 2.4. An intermediate corrected value, $(N_1)_{60}$, is calculated from the following equation:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{4.5}$$

where N_m is the measured standard penetration resistance, C_N is a correction factor for overburden pressure, C_E is the correction for hammer energy ratio (ER), C_B is a correction factor for borehole diameter, C_R is the correction factor for rod length, and C_S is the correction for samplers with or without liners. Suggested ranges of values for each of these correction factors are listed in Table 4-3. Selection of values for hazard screening are discussed below.

Because standard penetration resistance increases with increased effective overburden stress, an overburden stress correction factor is applied to adjust penetration resistance to a standard overburden pressure, P_a, of 100 kPa. This factor commonly is calculated from the following equation:

$$C_N = (P_a/\sigma'_{vo})^{0.5} \tag{4.6}$$

Equation 2.6 may be used for any system of units as long as both P_a and σ'_{vo} are reported in the same units. At shallow depths, C_N becomes large and is truncated at a maximum value of 2. The effective overburden pressure, σ'_{vo}, applied in this equation should be the overburden pressure that was effective at the time the SPT test was conducted. Even though the groundwater level may have changed after the SPT measurement, the correction of blow count requires the use of the effective pressures that were in effect at the time of drilling and testing. Calculating CSR (Equation 4.2), however, may require a different or higher groundwater level for conservatism.

Several other factors affect SPT results. One of the more important of these factors is the energy delivered to the SPT sampler. An energy ratio, ER, of 60% has generally been accepted as a reference value. The ER delivered by a particular SPT setup depends primarily on the type of hammer and anvil in the drilling system and on the method of hammer release. Approximate correction factors (C_E = ER/60%) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 4-3. These factors range from 0.5 to 1.2.

Corrections are also required for rod lengths less than 10 m and greater than 30 m, bore hole diameters outside the recommended range (65 mm to 125 mm), and sampling tubes without liners. Ranges of correction values for each of these variables are listed in Table 4-3. Careful documentation of drilling equipment and procedures, including measurement of ER, is required to select the most appropriate values for these correction factors. Even so, some uncertainty remains in the actual factors that should be applied.

4.3.2.1.2 Fines-Content Corrected Penetration Resistance, $(N_1)_{60cs}$

The NCEER Workshop (Youd and Idriss, eds., 1997) quantified the fines content correction to better fit the empirical data and to support computations with spreadsheets and other electronic computational aids. Seed et al. (1985) showed that for a given $(N_1)_{60}$, CRR increases with increased fines content. It is not clear, however, whether the CRR increase is caused by increased liquefaction resistance or by reduced penetration resistance. SPT resistance usually decreases with increased fines content because of concomitant increases in compressibility and decreases in permeability. Based on the empirical data available, Seed et al. developed CRR curves for various fines contents, as shown on Figure 4-3. These curves may be approximated by the following equations in routine practice to correct $(N_1)_{60}$ to $(N_1)_{60cs}$.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{4.7}$$

where α and β are coefficients determined from the following equations:

$\alpha = 0$	for $FC \leq 5\%$	(4.8a)
$\alpha = \exp[1.76 - (190/FC^2)]$	for $5\% < FC < 35\%$	(4.8b)
$\alpha = 5.0$	for $FC \geq 35\%$	(4.8c)

$\beta = 1.0$	for $FC \leq 5\%$	(4.9a)
$\beta = [0.99 + (FC^{1.5}/1000)]$	for $5\% < FC < 35\%$	(4.9b)
$\beta = 1.2$	for $FC \geq 35\%$	(4.9c)

where FC is the fines content measured from laboratory gradation tests on retrieved soil samples.

4.3.2.1.3. Magnitude Scaling Factors

To adjust the simplified base curve to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors called "magnitude scaling factors" (MSF). These factors are used to scale the simplified base curve upward or downward on the CSR versus $(N_1)_{60}$ plot. To illustrate the influence of magnitude scaling factor on calculated hazard, the equation for factor of safety (FS) against liquefaction can be written in terms of CRR, CSR, and MSF as follows:

$$FS = (CRR_{7.5}/CSR)MSF \tag{4.10}$$

The NCEER workshop reviewed MSF proposed by several investigators and recommended a range of factors as listed in Table 4-4. The MSF recommended by Idriss (Column 3, Table 4-4) forms the low end of the recommended range, and the MSF proposed by Andrus and Stokoe (Column 4, Table 4-4), form the high end of the range. The engineer should select MSF from within this range based on the degree of conservatism required for a given project. For screening liquefaction hazard, the more conservative MSF recommended by Idriss should be applied. The original factors of Seed and Idriss (1982) (Column 2, Table 4-4), although widely used in past engineering practice, have proven to be very conservative and are no longer recommended.

The following equations may be used to estimate MSF for routine applications:

Idriss MSF: $MSF = 10^{2.24}/M^{2.56} = (M_w/7.5)^{-2.56}$ (4.11)

Andrus and Stokoe MSF: $MSF = (M_w/7.5)^{-3.3}$ (4.12)

Table 4-3 Corrections to SPT (modified from Skempton, 1986, by Robertson and Wride (1997)

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma'_{vo})^{0.5}$; $C_N \leq 2$
Energy ratio	Donut Hammer Safety Hammer Automatic-Trip Donut-Type Hammer	C_E	0.5 to 1.0 0.6 to 1.2 0.8 to 1.3
Borehole diameter	65 mm to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Rod length	3 m to 4 m 4 m to 6 m 6m to 10 m 10 to 30 m >30 m	C_R	0.75 0.85 0.95 1.0 <1.0
Sampling method	Standard sampler Sampler without liners	C_S	1.0 1.1 to 1.3

4.3.2.2 Analysis and Classification

The above criteria may be applied to calculate the factor of safety against liquefaction (Equation 4.1) for granular layers for which standard penetration tests and grain-size data are available. If the factor of safety for all layers is greater than 1.5, the site can be classed as low liquefaction hazard and low priority for further investigation. If one or more layers are characterized by a factor of safety of 1.5 or less, the screening continues to the next step as indicated in Figure 2-1. Where reported standard penetration data are inadequate to calculate factors of safety for all granular layers at depths of 15 m or less, and where cone penetration or other data are unavailable for calculation of liquefaction resistance, the site should be classed as potentially liquefiable and prioritized for further investigation using geologic and other criteria, such as the importance of the structure. If cone penetration data are available, the procedures in the following section may be used to evaluate factors of safety against liquefaction for granular layers at the site.

Table 4-4 Magnitude Scaling Factors for Liquefaction Hazard Evaluation

Mw (1)	Seed and Idriss (1982) (2)	Idriss (written commun.) (3)	Andrus and Stokoe (1997) (4)
5.5	1.43	2.20	2.8
6.0	1.32	1.76	2.1
6.5	1.19	1.44	1.6
7.0	1.08	1.19	1.25
7.5	1.00	1.00	1.00
8.0	0.94	0.84	0.8
8.5	0.89	0.72	0.65

4.3.3 Cone Penetration (CPT) Resistance

Although not as commonly used as SPT, the cone penetration test (CPT) is becoming a major tool for delineating soil stratigraphy and for conducting preliminary evaluations of liquefaction resistance. The superior capability of the CPT to detect stratigraphic layering, compared to the other tools listed in Table 4-2, makes the CPT particularly advantageous for site reconnaissance investigations. Criteria have been developed for calculating liquefaction resistance (CRR) directly from CPT data (Youd and Idriss, 1997). These criteria may be applied in practice--provided adequate samples are retrieved, preferably using SPT procedures--to verify the soil types and liquefaction resistances assigned.

Figure 4-5 shows the primary chart used for determining liquefaction resistance from CPT data for clean sands. The chart shows CSR plotted against corrected and normalized CPT resistance, q_{c1N}, form sites where liquefaction effects were or were not observed following past earthquakes. A CRR curve separates regions of the plot with data indicative of liquefaction from regions with data indicative of nonliquefaction. This chart is valid for magnitude 7.5 earthquakes and clean, sandy materials. Dashed curves, showing approximate shear strain potential, γ_{ℓ}, as a function of q_{c1N}, are also drawn on the figure to emphasize the fact that cyclic shear strain and ground deformation potential at liquefiable sites decrease as penetration resistance increases.

The CRR curve plotted in Figure 4-5 may be approximated by the following equations (Robertson and Wride, 1997):

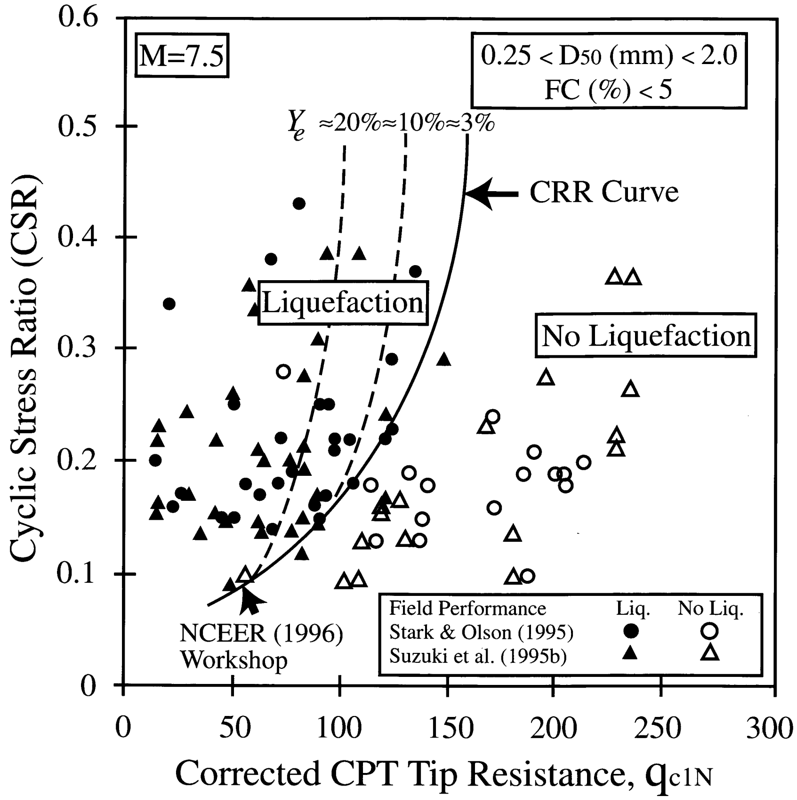


FIGURE 4-5 Curve Recommended for Calculating CRR from CPT Data along with Empirical Liquefaction Data (after Youd and Idriss, 1997)

$$CRR_{7.5} = 0.833[(q_{c1N})_{cs}/1000] + 0.05 \qquad \text{for } (q_{c1N})_{cs} < 50 \qquad (4.13a)$$

$$CRR_{7.5} = 93\ [(q_{c1N})_{cs}/1000]^3 + 0.08 \qquad \text{for } 50 \leq (q_{c1N})_{cs} \leq 160 \qquad (4.13b)$$

$$\text{Nonliquefiable} \qquad \text{for } (q_{c1N})_{cs} > 160$$

where $(q_{c1N})_{cs}$ is the equivalent clean-sand cone penetration resistance normalized to one atmosphere of pressure (approximately 100 kPa). Procedures for correcting penetration measurements in silty sands to clean-sand values are given in subsequent paragraphs.

4.3.3.1 Data Required

The data required for calculating liquefaction resistance from CPT data are logs of cone tip, q_c, and sleeve friction, f_s. The cone used in the site investigation should have been a standard electrical type with a cross-sectional area of 10 cm^2 and a 100 mm-long friction sleeve. Specifications for cone penetration test equipment and procedures are given in ASTM D 3441. Because the CPT equipment and procedures are much less variable than those for the SPT, fewer corrections are required.

Nevertheless, corrections are required for both overburden pressure and grain characteristics. Those corrections are described below.

4.3.3.1.1 Normalized and Corrected CPT Resistance, q_{c1N}

The equation for correcting and normalizing CPT data to one atmosphere of overburden pressure is

$$q_{c1N} = C_Q(q_c / P_a) \tag{4.14}$$

where

$$C_Q = (P_a/\sigma'_{vo})^n \tag{4.15}$$

q_{c1N} is the normalized and corrected CPT resistance; C_Q is a normalizing factor; and P_a is 100 kPa or approximately atmospheric pressure reported in the same units as the measured cone penetration resistance, q_c. A maximum C_Q value of 2 is generally applied to correct CPT data at shallow depths. The value of the exponent, n, in Equation 4.15 depends on grain characteristics of the soil and ranges from 0.5 for clean sands to 1.0 for clays. Although somewhat complicated, the following equations and procedures for correcting tip resistance can be readily programmed into electronic aids such as spreadsheets; commercial software that incorporate these techniques are available.

The CPT friction ratio (sleeve resistance divided by cone tip resistance) generally increases with increasing fines content and soil plasticity. Robertson and Wride (1997) suggest that corrections for grain characteristics can be estimated from analysis of q_c and f_s data: Figure 4-6 is a plot of CPT soil types as a function of normalized cone tip resistance, Q, and normalized friction ratio, F. Boundaries between CPT soil types 2 through 7 were approximated as concentric circles by Jeffries and Davies (1993). They defined the radius of such circles as the soil behavior type index, I_c, which can be calculated from the following equation:

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \text{Log } F)^2]^{0.5} \tag{4.16}$$

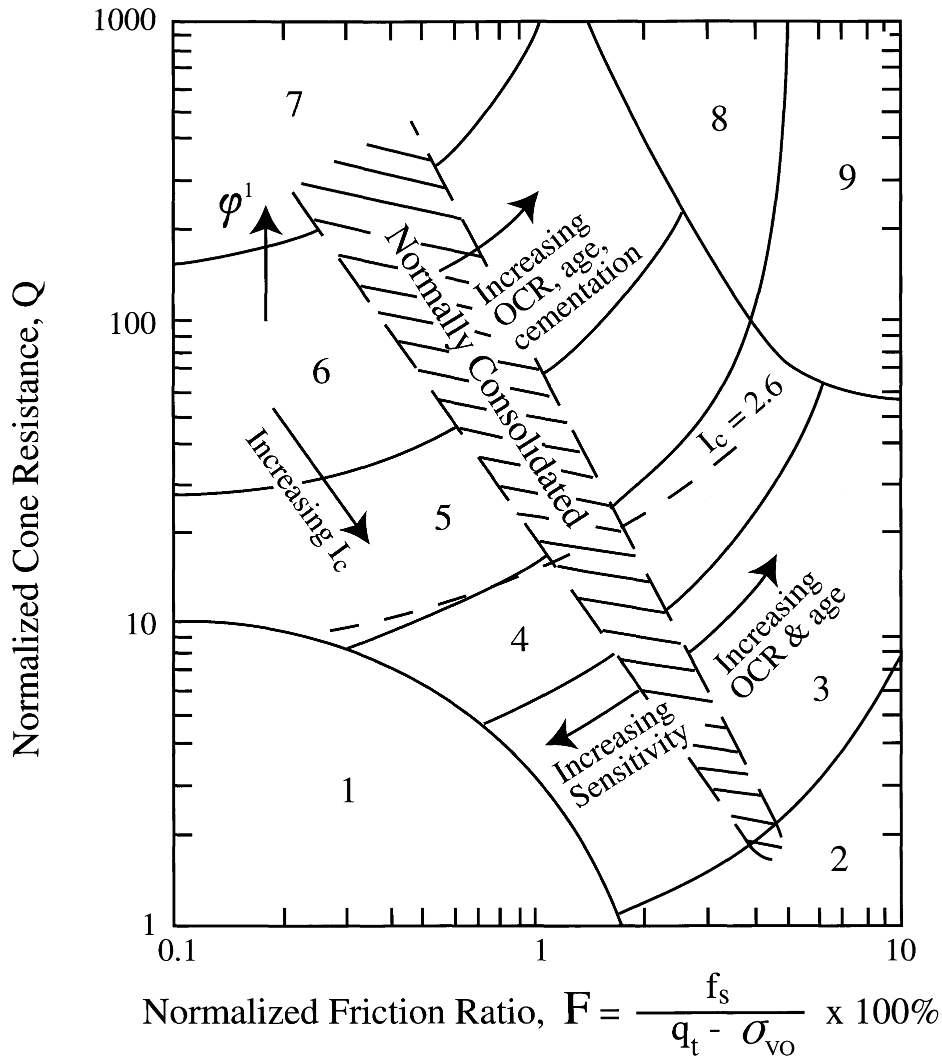
where

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \tag{4.17}$$

and

$$F = [f_s/(q_c - \sigma_{vo})] \times 100\% \tag{4.18}$$

The soil-type chart in Figure 4-6 was developed using an exponent, n, of 1.0, which is the appropriate value for clayey type soils. For clean sands, however, an exponential value of 0.5 is more appropriate, and a value intermediate between 0.5 and 1.0 would be appropriate for silts and silty sands.



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

**Heavily overconsilidated or cemented*

FIGURE 4-6 CPT-Based Soil Behavior Type Chart (modified from Robertson, 1990)

The first step in calculating I_c is to differentiate soil types characterized as clays from soil types characterized as sands and silts. This differentiation is performed by assuming an exponent, n, of 1.0 (characteristic of clays) to calculate Q from Equation 4.17 and I_c from Equation 4.16. If the calculated I_c is greater than 2.6, the soil is classed as clayey and most likely is too clay rich or plastic to liquefy and the analysis is complete. Soil samples from the site should be analyzed, however, to confirm the clayey soil type. The criteria in Table 4-1 might be applied to confirm that the soil is indeed nonliquefiable.

If the calculated I_c is less than 2.6, the soil is most likely granular in nature and Q should be recalculated from Equation 4.17 using an n-value of 0.5 and I_c recalculated from Equation 4.16. C_Q should also be recalculated with an exponent of 0.5 (Equation 4.15) and q_{c1N} (calculated from Equation 4.14). If the recalculated I_c is less than 2.6, the soil can be classed as nonplastic and granular, and this I_c used to estimate the clean-sand equivalent normalized CPT resistance, $(q_{c1N})_{cs}$, and the liquefaction resistance as noted below.

If the recalculated I_c is greater than 2.6, however, the soil is likely to be very silty and possibly plastic. In this instance, Q and q_{c1N} should be recalculated using an intermediate exponent, n, of 0.7 and I_c and q_{c1N} for the silty soil recalculated as before. These intermediate values of q_{c1N} and I_c are then used to calculate the liquefaction resistance. Again, data from soil samples should be evaluated to verify the soil type and the liquefiability of the sediment.

4.3.3.1.2 Clean Sand Equivalent Corrected and Normalized CPT Resistance, $(q_{c1N})_{cs}$

To correct the normalized penetration resistance, q_{c1N}, of silty sands to an equivalent clean sand value, $(q_{c1N})_{cs}$, for use in liquefaction resistance calculations, the following relationships are applied:

$$(q_{c1N})_{cs} = K_c q_{1cN} \qquad (4.19)$$

where K_c is the CPT correction factor for grain characteristics. K_c is defined by the following equations (Robertson and Wride, 1997):

$$\text{For } I_c \leq 1.64 \quad K_c = 1.0 \qquad (4.20a)$$

$$\text{For } I_c > 1.64 \quad K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63\, I_c^2 + 33.75\, I_c - 17.88 \qquad (4.20b)$$

With appropriate values for I_c and K_c, Equation 4.19 is used to calculate $(q_{c1N})_{cs}$ and that value substituted into Equation 4.13 to calculate $CRR_{7.5}$. To adjust CRR to magnitudes smaller or larger than 7.5, the calculated $CRR_{7.5}$ is multiplied by an appropriate magnitude scaling factor. The same magnitude scaling factors (Table 4-4) are used with CPT and SPT data.

4.3.3.1.3 Correction of CPT Resistance for Thin Sand Layers

Theoretical as well as laboratory studies indicate that cone resistance is influenced by softer or stiffer soil layers above or below the cone tip. As a result, the CPT will not usually measure the full penetration resistance in thin sand layers sandwiched between layers of softer soils. The distance to which cone tip resistance is influenced by an approaching interface increases with stiffness of the stiff layer. In soft clays or loose sands, the distance of influence can be as small as 2 to 3 cone diameters. In stiff clays or dense sands, the distance of influence may be as large as 20 cone diameters. Thus care should be taken when interpreting cone resistance of thin sand layers sandwiched between silt or clay layers with lower penetration resistances. Based on a simplified elastic solution, Vreugdenhil et al. (1994) developed a procedure for estimating the full cone penetration resistance of thin, stiff layers contained within softer strata. Based on this model, Robertson and Fear (1995) suggest a correction factor for cone resistance, K_H, as a function of layer thickness as shown in Figure 4-7. The correction applies only to thin, stiff layers embedded within thick, soft layers. The equation for evaluating the correction factor, K_H, is

$$K_H = 0.5\,[(H/1{,}000) - 1.45]^2 + 1.0 \qquad \text{for } H<1{,}400 \text{ mm} \tag{4.21}$$

and the equation for correcting the measured thin-layer tip resistance, q_{cA}, to an equivalent thick-layer resistance, q_c^*, is

$$q_c^* = (K_H)(q_{cA}) \tag{4.22}$$

4.3.3.2 Analysis and Classification

Cone penetration data are normally digitized at 100 mm intervals. Thus direct application of the equations in this section will typically provide a calculated factor of safety against liquefaction within granular layers at 100 mm intervals. Because layers less than about 300 mm thick are not very significant and because layers thinner than this may not be accurately evaluated with CPT criteria, isolated liquefiable layers less than 300 mm thick may disregarded in assessing liquefaction hazard. As an alternative to evaluating each digitized point, averages may be taken over 300 mm to 500 mm intervals in identifying significant liquefiable layers. Where granular layers thicker than 300 mm are characterized with a factor of safety less than 1.3, the site should be classed as potentially liquefiable and the screening procedure proceeds to the next step as indicated in Table 2.1. If the factor of safety for all layers is greater than 1.3, the site can be classed as low liquefaction hazard and low priority for further investigation.

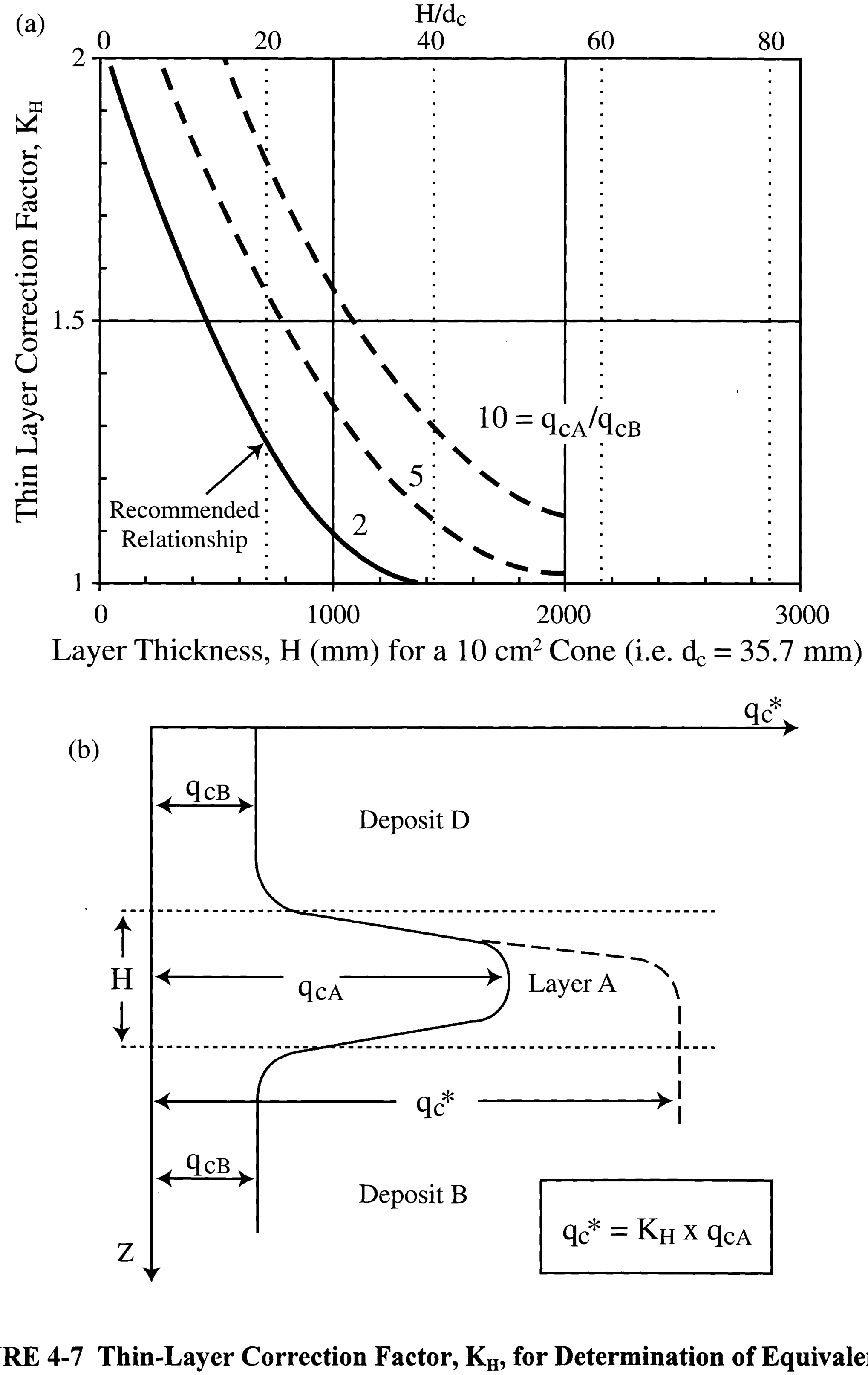


FIGURE 4-7 Thin-Layer Correction Factor, K_H, for Determination of Equivalent Thick-Layer CPT Resistance (modified from Robertson and Fear, 1995)

SECTION 5
ASSESSMENT OF GROUND DISPLACEMENT HAZARD

Liquefaction by itself is not a cause of bridge damage. Structural damage occurs when liquefaction induces intolerable ground displacements or deformations or loss of foundation bearing strength. Displacements occur as a consequences of embankment instability, inertially induced deformation, lateral spread, or ground settlement. Analyses of embankment instability and inertial deformation require complex and sophisticated computational procedures and engineering expertise beyond that required for routine screening analyses. Thus only general procedures are given for these evaluations in the following paragraphs. Geotechnical specialists should be consulted where applications of these procedures are required.

Insufficient case history data have been compiled or analyzed to develop quantitative relationships between horizontal ground displacement and bridge damage. Reviews of past bridge performance indicates that most bridges can withstand up to 100 mm of lateral ground displacement without significant damage (Youd, 1993). Displacements of 100 mm or less are generally accommodated through shear or compression of soils rather than through bridge deformation. Even relatively weak bridges, such as light timber structures, have withstood 100 mm of ground displacement without significant distress to the bridge structure. Stronger bridges should be able to withstand much larger ground displacements without distress. Performance criteria have not been developed for bridge damage or distress as a consequence of ground displacements greater than 100 mm. Thus for this screening guide, estimated lateral ground displacements less than 100 mm are considered nonhazardous, while displacements greater than 100 mm are considered potentially hazardous.

Similarly, vertical ground displacements or ground settlements less than 25 mm seldom cause damage to bridges supported by shallow foundations or 100 mm for bridges supported on deep foundations. Thus for screening purposes, vertical displacements less than 25 mm for shallow foundations and 100 mm for deep foundations are also classed as non-hazardous, while vertical displacements greater than these values are classed are classed as potentially hazardous.

5.1 Embankment or Slope Instability

Liquefaction of soil layers temporarily reduces the shear strength of contractive soil. If strength loss beneath an embankment or a steep slope, such as a river bank, reduces shear resistance to a level less than that required for static equilibrium, catastrophic flow failure may ensue. Such failures are generally associated with large ground displacements that could cause fracture or displace bridge foundation elements or collapse approach embankments.

5.1.1 Data Required

The primary data required for analysis of slope stability include the geometry of the slope, soil stratigraphy with appropriate density and strength properties for each soil layer. An estimate of the post-liquefaction or residual shear strength is also required for each liquefiable layer. Most of these

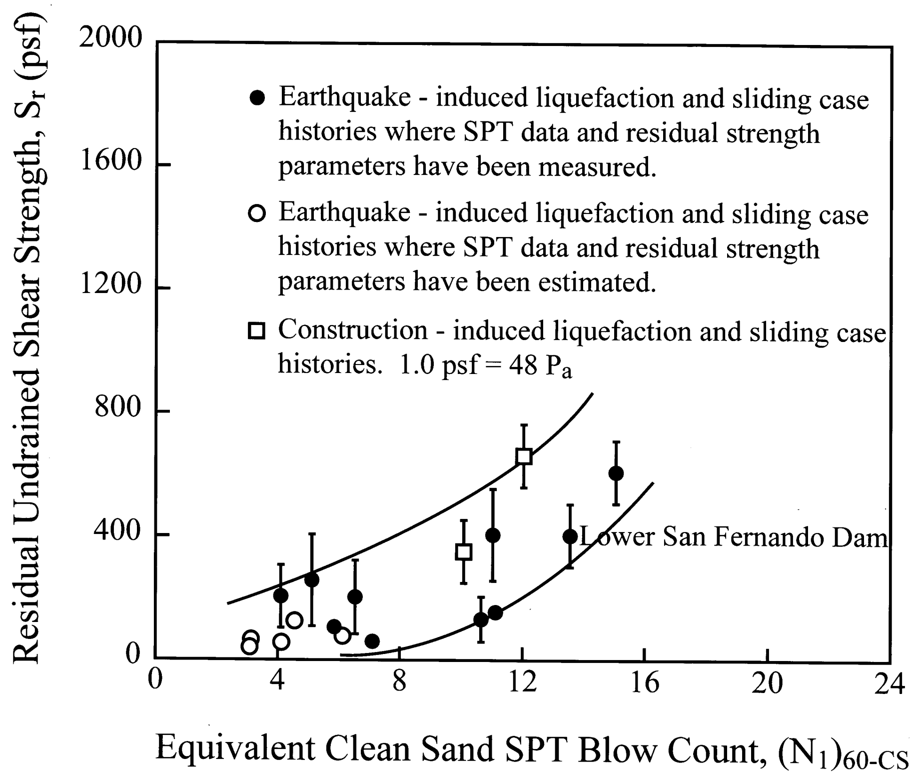


Figure 5-1 Empirical Relationship Between Residual Shear Strength and $(N_1)_{60cs}$ (After Seed and Harder, 1990)

properties are not determined as part of routine foundation investigations. Generally, additional drilling, testing, and data interpretation are required for stability analyses. The present state-of-the-art is to estimate residual strengths for liquefiable layers from corrected standard penetration resistance, $(N_1)_{60}$, using criteria (Figure 5-1) developed by Seed and Harder (1990). For critical structures, such as large or heavily traveled bridges, the lower bound of the undrained residual strength plot (Figure 5-1) is commonly used. For less critical structures rivers, residual strengths may be selected from the mid to lower part of the plot.

5.1.2 Analysis and Classification

The stability of a slope or embankment can be checked using standard limit-equilibrium procedures. Commercial computational programs are available to aid this analysis. The analysis is conducted with undrained residual strengths assigned to all liquefiable layers and conservative estimates of drained strengths and undrained strengths for other materials above and below the watertable, respectively. For the stability analysis, only static (gravitational) forces are applied to the embankment or slope. Inertial forces generated by the earthquake are usually neglected at this stage of the investigation. If the calculated static factor of safety (FS) is equal to or less than 1.1, the site could become unstable during or after an earthquake, leading to large soil deformations. Such deformations would likely generate large embankment or slope displacements that could adversely affect the integrity of the bridge foundation. Thus if the stability analysis indicates a static factor of safety equal to or less than 1.1, the bridge site is classed as potentially hazardous and prioritized for further investigation and possible remediation.

If the static factor of safety is greater than 1.1, the embankment is classed as stable against catastrophic slope failure, but the embankment may still undergo damaging deformation due to soil softening. In such instances, the evaluation continues to the next step in the analysis (Figure 2.1).

5.2 Analysis of Embankment or Slope Deformation

Damaging ground deformations may occur within or beneath embankments or slopes as a consequence of liquefaction, even though the site may be stable against catastrophic flow failure. Such deformations occur as a consequence of soil softening and yielding due to liquefaction and the inertial forces generated by the earthquake. In these instances, cyclic mobility and limited strains within liquefied layers may lead to ground deformations and displacements that could damage the bridge structure. Analyses of embankment or slope deformation at liquefiable sites is complicated because of the complex nature of constitutive relations for liquefied soils. In particular, stress-strain relations are very complex for moderately dense or dilative soils that may deform under either undrained or partially drained conditions. The nonhomogeneity of natural soils also complicates the characterization of soil properties. The erratic nature and general unpredictability of inertial loads generated by the earthquake additionally complicates the analysis. Thus deformation analyses are generally performed by specialists in geotechnical earthquake engineering.

5.2.1 Data Required

All of the data required for analysis of slope stability analyses (Section 5.1) also are required for analysis of embankment or slope deformation. In addition, a strong motion accelerogram, or set of accelerograms, is required. These accelerograms should be representative of ground motions expected at the site for earthquakes expected in the region. Soil properties required for the analysis include stiffness and damping values for each soil layer and constitutive relations between stresses and strains and between strains and pore-pressure changes. Stiffness is usually couched in terms of small-strain shear modulus, G_{max}, and modulus and damping relationships that are functions of shear strain, soil plasticity, and over-consolidation ratio. Some of the more sophisticated analysis packages apply finite element analysis and require constitutive relationships for the liquefied soil. As noted above, these relationships tend to be complex and difficult to define and quantify for in-place soils at natural sites. The more simple analyses assume plastic soil deformation and simple stress-strain behavior that can be estimated from soil strength data. Although perhaps not as exact, the simpler procedures are much easier to apply and require less soil testing and analysis. Even with the simplified procedures, specialized expertise is required, however, to properly perform the analysis and interpret the results. Because this level of expertise exceeds that of typical practicing geotechnical engineers, outside specialists should be consulted or employed to perform embankment deformation analyses.

5.2.2 Analysis and Classification

No single procedure has gained widespread acceptance for analysis of embankment or slope deformations at liquefiable sites. The most widely used analyses are based on the simplified sliding-block model of Newmark (1965). For example, the procedures developed by Makdisi and Seed (1978) and by Byrne (1991) use the sliding-block mechanism. The procedure requires ground response evaluation for critical points in the slope or embankment. Only acceleration pulses with magnitudes greater than a critical or yield acceleration can activate the sliding block and generate slope displacement. Displacement continues for the short period of time while inertial and kinetic forces are sufficient to sustain movement. Once movement generated by the acceleration pulse is arrested by soil shear resistance, no additional displacement occurs until a subsequent acceleration pulse exceeds the yield acceleration. The incremental displacements generated by each acceleration pulse are then summed to yield an estimate of total slope displacement.

The yield acceleration is a function of the factor of safety calculated for static slope stability (described in Section 5.1.2). For a slope with a factor of safety of 1.0, the yield acceleration is zero and all down-slope horizontal accelerations generated by an earthquake induce soil yielding and slope deformation. As the static factor of safety increases, the yield acceleration also increases, reducing the number and widths of acceleration pulses that exceed the yield level. These reductions of mobilizing forces markedly decrease the amount of calculated slope displacement. Past experience and analytical calculations indicate that slope deformations are generally negligible for sites with an adequate static factor of safety. Adequate factors of safety that generally reduce displacements to acceptable limits (less than 100 mm) are approximately 1.5 for magnitude 6.5 earthquakes, 2.0 for magnitude 7.5 earthquakes, and 2.5 for magnitude 8.5 earthquakes. For screening applications, if the static factor of safety against slope failure is greater than these values for the given earthquake magnitudes, the embankment or slope can be classed as minimally deformable and non-hazardous, and the screening analysis proceeds to the next step.

If the static factor of safety is less than the above limits, a dynamic deformation analysis should be conducted. As noted above, expert assistance will usually be required to conduct this analysis. The result of the analysis will be a tabulation or contour plot of predicted slope displacements at various points on and within the slope or embankment. Very little guidance is available from analysis or case history observations on amounts of ground displacement bridge structures can withstand without unacceptable distress to the foundation or bridge structure. Such thresholds are a function of several factors including bridge strength, foundation configuration, and the distribution of ground displacements acting on the structure. As noted in the introduction to Section 5, common highway bridges can generally withstand up to 100 mm of ground displacement without significant distress to abutments or foundation elements. Using the 100 mm displacement as a threshold for screening yields the following criterion. If the predicted slope deformations at points of intersection with the bridge structure exceed 100 mm, the bridge site should be classed as potentially hazardous and prioritized for further investigation and possible remediation. If predicted slope displacements are less than 100 mm, the site can be classed immune to local slope deformation, and the screening evaluation proceeds to the next step (Figure 2.1).

5.3 Lateral Spread Displacement

If steep slopes and embankments are stable against slope failure and excess deformation, the next mode of ground displacement to be evaluated is liquefaction-induced lateral spread. This type of failure induces lateral displacement of natural or filled ground down gentle slopes or toward free faces such as incised river and stream channels. Lateral spread displacements generate lateral earth pressures that press against abutments and bridge foundation and may fracture and displace these elements. As noted in Section 1, lateral spread has been the most common cause of liquefaction-induced bridge damage during past earthquakes. Nearly all of this damage was due to displacement of flood plain or fill deposits toward river channels or other incised water bodies.

For routine analyses such as hazard screening, the empirical procedure of Bartlett and Youd (1995) has been widely used for estimating lateral ground displacements. This procedure is easy to apply and only requires information developed during typical high-quality foundation investigations. From a multiple linear regression (MLR) analysis of lateral spread case history data, Bartlett and Youd (1995) developed two empirical equations for predicting lateral ground displacement, one for free faces, such as incised river channels and the other for gently sloping ground conditions.

For free-face conditions:

$$\begin{aligned} \mathrm{LOG}\ D_H = &- 16.3658 + 1.1782\ M - 0.9275\ \mathrm{LOG}\ R - 0.0133\ R + 0.6572\ \mathrm{LOG}\ W \\ &+ 0.3483\ \mathrm{LOG}\ T_{15} + 4.5270\ \mathrm{LOG}\ (100 - F_{15}) - 0.9224\ D50_{15} \end{aligned} \tag{5.1a}$$

and for ground slope conditions:

$$\begin{aligned} \mathrm{LOG}\ D_H = &- 15.7870 + 1.1782\ M - 0.9275\ \mathrm{LOG}\ R - 0.0133\ R + 0.4293\ \mathrm{LOG}\ S \\ &+ 0.3483\ \mathrm{LOG}\ T_{15} + 4.5270\ \mathrm{LOG}\ (100 - F_{15}) - 0.9224\ D50_{15} \end{aligned} \tag{5.1b}$$

where D_H is the estimated lateral ground displacement, in meters; M is the estimated moment magnitude of the earthquake; R is horizontal distance from the seismic energy source, in kilometers; T_{15} is the cumulative thickness in meters of saturated granular layers with corrected blow counts, (N1)60, less than 15; F_{15} is the average fines content (fraction of sediment sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent; $D50_{15}$ is the average mean grain size in granular layers included in T_{15}, in millimeters; S is the ground slope, in percent; and W is the free-face ratio defined as the height (H) of the free face divided by the distance (L) from the base of the free face to the point in question, in percent. Because of the empirical nature of Equations 5.1a and 5.1b, these relationships are valid only for the limited range of values for each independent variable incorporated in the compiled data base. The ranges for these limiting values are listed in Table 5-1. Extrapolation beyond the limits will lead to uncertain results.

Table 5-1 Ranges of Values for Independent Variables for Which Lateral Spread Equations Are Verified by Case-History Observations (after Bartlett and Youd, 1992)

Input Factor	Range of Values in Case History Database
Magnitude	$6.0 < M < 8.0$
Free-Face Ratio	$1.0\% < W < 20\%$
Ground Slope	$0.1\% < S < 6\%$
Thickness of Loose Layer	$0.3\ m < T_{15} < 12\ m$
Fines Content	$0\% < F_{15} < 50\%$
Mean Grain Size	$0.1\ mm < D50_{15} < 1\ mm$
Depth to Bottom of Section	Depth to Bottom of Liquefied Zone < 15 m

Bartlett and Youd (1995) note that there is some statistical uncertainty in D_H calculated from Equations 5.1a and 5.1b, even when the variables are within the ranges noted in Table 5-1. Doubling the calculated D_H, however, will yield estimates with a high probability of not being exceeded. Thus for screening applications, calculated values of D_H should be doubled to assure adequate conservatism.

5.3.1 Data Required

The data required to apply the Bartlett and Youd procedure is that necessary to define each of the independent variables in the equations. Requirements for each variable are briefly discussed below. More guidance, detailed explanations, and example calculations for selecting values for each variable are given by Youd (1993b).

Earthquake Magnitude, M. The moment magnitude, M, is specified for calculation of D_H from Equations 5.1a and 5.1b. This same magnitude is generally specified for calculation of liquefaction resistance as noted in Section 4.3.1. Magnitudes estimated from other scales may be substituted, however, within the limits noted in Section 4.3.2.2.

Seismic Source Distance, R. The seismic source distance, R, is defined as the horizontal distance, in kilometers, from the site in question to the nearest point on a surface projection of the seismic source zone. For many localities in the western U.S., the distance from known active faults may be used as an estimate for R. For regions where seismicity is associated with broad source zones rather than discrete faults, the distance R is measured from the nearest point on or within the source zone boundary. Minimal values for R for use with either faults or source zones are listed in Table 5-2. For earthquakes with magnitudes less than 6, epicentral distances provide adequate estimates for R. For magnitudes greater than 6 a single point--the epicenter--is not adequate to represent a large fault

TABLE 5-2 Minimum Values of R for Use in Equations 5.1 (after Bartlett and Youd, 1992)

Magnitude M_w	Minimum Value of R km
6.0	0.5
6.5	1
7.0	5
7.5	10
8.0	20-30

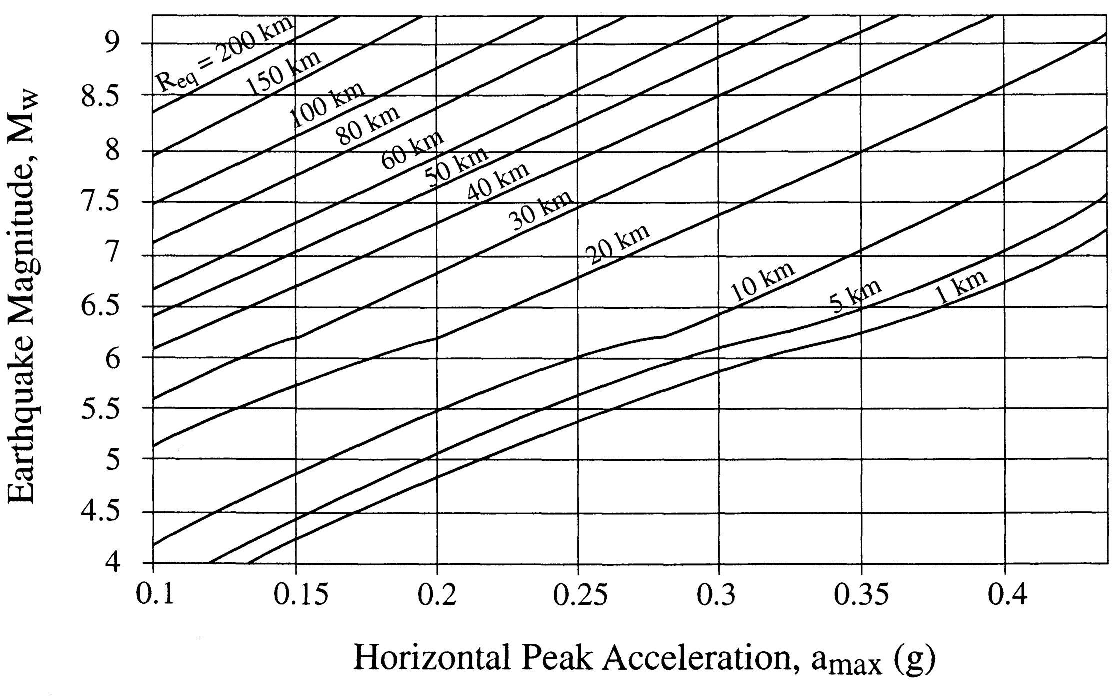


Figure 5-2 Curves for Determining Equivalent Source Distance, R_{eq}, From Magnitude and Estimated a_{max} (After Bartlett and Youd, 1992)

rupture zone. Thus distance to the nearest bound on a surface projection of the source zone should be used rather than epicentral distance for large events.

For earthquakes in the eastern U.S. or for sites underlain by soft soils where amplification of ground motions may be large, an equivalent distance, R_{eq}, should be used in place of R. R_{eq} is estimated from M and a_{max} as follows: An estimated a_{max} is developed for the site based on local ground response analyses or use of amplification factors or ratios as described in Section 4.3.2.2. The a_{max} determined, however, must be a mean-expected value rather than a more conservative value, such as an a_{max} with mean plus one standard deviation probability of occurrence, as is often used for conservative engineering design. Use of a_{max} values that are more conservative than the mean will yield overly conservative estimates of D_H. The mean a_{max} is then plotted against magnitude on Figure 5-2, and R_{eq} is interpolated from the curves on the plot. That R_{eq} is then used in Equations 5.1a or 5.1b to estimate D_H. The R_{eq} procedure is only valid for a_{max} less than 0.4 g and magnitudes less than 8. Extrapolation beyond these values will lead to uncertain predictions.

Thickness of loose granular sediment, T_{15}. T_{15} is an estimate of the thickness of loose granular sediment at a liquefiable site. Bartlett and Youd (1995) define T_{15} as the thickness of liquefiable granular sediments in a soil profile characterized by an $(N_1)_{60}$ equal to or less than 15. Where there are distinct lithologic changes in granular sediments, such as distinct layers of clean and silty sand, separate displacement calculations should be made for each layer and the displacements for the all the layers summed to provide the final estimate of D_H.

Average fines content, F_{15}. F_{15} is defined as the average fines content (i.e., the percent of material passing a No. 200 sieve) from all samples taken from a layer characterized by a thickness, T_{15}.

Average mean-grain size, $D50_{15}$. Bartlett and Youd (1995) characterized the coarseness of a layer by the parameter, $D50_{15}$, which is defined as the average mean-grain size of materials included in layer T_{15}. This variable is determined by averaging mean-grain sizes measured in each layer characterized by a thickness T_{15}.

Ground Slope, S. The ground slope, S, corresponds to the standard engineering definition of slope, or the rise of elevation over the horizontal run of the slope, expressed in percent. S is the average ground slope and generally disregards minor topographic rises and depressions.

Free-Face Ratio, W. The free-face ratio is defined as the height, H, of a free face divided by the distance, L, from the point in question to the base of the free face:

$$W = (H/L)(100), \text{ in percent} \tag{5.2}$$

The height, H, is commonly determined by subtracting the elevation at the base of a depression, such as a river bottom or the toe of a fill, from the elevation at the crest of the bank or fill. The distance, L, is measured from the base or toe of the free face to the locality in question.

5.3.2 Analysis and Classification

Estimates of D_H are calculated from Equations 5.1a and 5.1b, with input variables determined as noted in the previous section. Because of statistical uncertainty, the calculated D_H should be doubled to provide adequate conservatism for screening applications. If the conservatively estimated displacement is less than 100 mm, the site can be classed as not susceptible to significant lateral ground displacement. If the estimated displacements are 100 mm or greater, the bridge should be classed as possibly hazardous and high priority for further investigation. If insufficient data are available to apply Equations 5.1a or 5.1b or if the available data fall outside the limiting ranges in Table 5-1, the bridge should also be classified as possibly hazardous and prioritized for further investigation.

5.4 Analysis of Ground Settlement

Earthquake shaking is an effective compactor of granular soils--dry, moist, or saturated--leading to vertical ground displacement or ground settlement. Laboratory studies show that compaction is enhanced by liquefaction of granular materials (Lee and Albaisa, 1974). Tokimatsu and Seed (1987) compiled data from laboratory tests and case histories of earthquake-induced ground settlements and developed a simplified empirical method for estimating settlement. That procedure is recommended here for estimating liquefaction-induced settlement for hazard screening purposes. Ishihara and Yoshimine (1992) also developed a technique for estimating ground settlement; the predicted results, however, are usually comparable to those determined from the Tokimatsu and Seed procedure.

The premise of the Tokimatsu and Seed procedure is that earthquake shaking generates cyclic shear strains that compact granular soils, causing volumetric strain. Where drainage cannot occur rapidly, the tendency to compact also generates transient pore water pressures that prevent immediate decrease in volume. However, as pore pressures dissipate, the layer consolidates, producing volumetric strain and ground settlement. Tokimatsu and Seed show that the induced volumetric strains are primarily a function of amplitude of the cyclic shear strains generated by the earthquake and the initial relative density of the sand. The cyclic shear strains are a function of the cyclic stress ratio (CSR), relative density, and earthquake magnitude. They correct the cyclic stress ratio for magnitude by dividing the CSR by an appropriate magnitude scaling factor from Table 4-4. Relative density was estimated directly from corrected penetration resistance, $(N_1)_{60}$. Figure 5-3 is a synthesis diagram developed by Tokimatsu and Seed from available laboratory tests data and field observations of earthquake-induced settlements in clean sands. They recommend use of this diagram to estimate volumetric strains from magnitude-corrected CSR and $(N_1)_{60}$ determined for the particular site and sand layer. That volumetric strain is then multiplied by the layer thickness, assuming one-dimensional consolidation, to estimate the change in thickness. The changes in thickness from all layers at the site are then summed to provide an estimate of the total settlement.

The Tokimatsu and Seed procedure was developed for clean sands. For screening purposes the procedure may be applied to silty sands by converting the blow count to an equivalent clean sand value, $(N_1)_{60cs}$, using Equations 4.7 to 4.9.

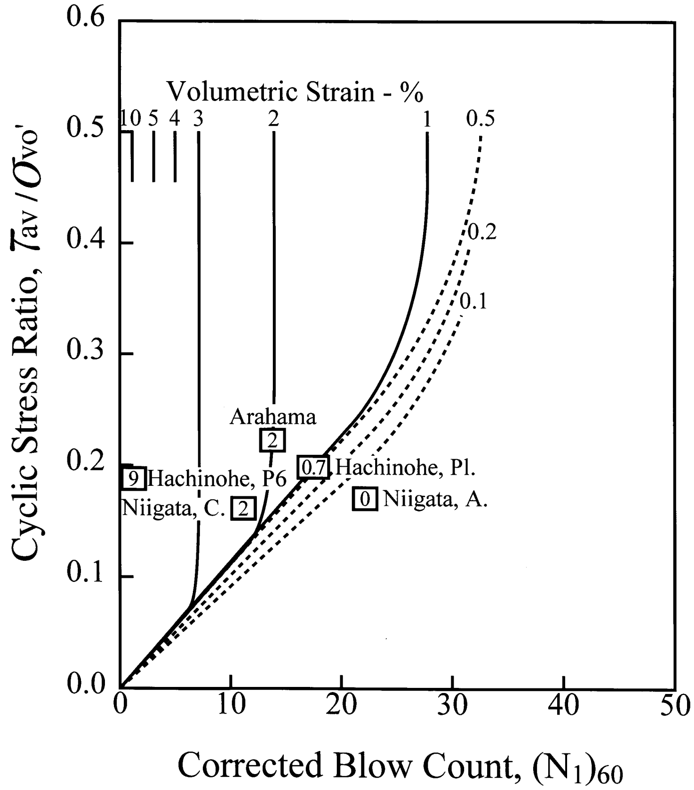


Figure 5-3 Curves for Estimating Volumetric Strain at Liquefiable Sites (After Tokimatsu and Seed, 1987)

5.4.1 Data Required

The data required to estimate ground settlement by the Tokimatsu and Seed (1987) procedure are the same as those required to apply the simplified procedure for calculating liquefaction resistance (Section 4.3.2), plus estimates of layer thicknesses. Layer thicknesses can be estimated from borehole or CPT logs or interpolated from stratigraphic cross-sections.

5.4.2 Analysis and Classification

The analysis of ground settlements consists of estimating CSR for the site and $(N_1)_{60cs}$ values for each granular layer beneath the site. These factors are then used with Figure 5-3 to estimate volumetric strains for each layer. The increment of ground settlement for each layer is calculated by multiplying volumetric strain by the layer thickness. The total settlement is then calculated by summing up the incremental settlements for all of the layers.

Design criteria for most structures on shallow foundations limit ground settlements to 25 mm or less. Structures on deep foundations can generally withstand settlements of 100 mm or more, so long as structural loads are transferred downward to competent strata and the foundation itself does not settle more than 25 mm. These limits are applicable to settlements caused by liquefaction as well as those caused by static consolidation of compressible layers. Based on this general guidance, the following conservative criteria are recommended for hazard screening for highway bridges: (1) Bridges supported on shallow foundations may be classed as nonvulnerable to settlement damage if estimated ground settlements are 25 mm or less. In this instance, the screening analysis proceeds to the final step of assessing foundation bearing capacity (Section 5.5). If predicted settlements are greater than 25 mm, the structure should be classed as possibly hazardous and prioritized for further investigation. (2) For structures supported by deep foundations, if predicted settlements are 100 mm or less, the structure may be classed as nonvulnerable to ground settlement, and the load capacity of the foundation should be evaluated (Section 5.5). If predicted settlements are greater than 100 mm, the structure should be classed as possibly hazardous and prioritized for further investigation.

5.5 Bearing Capacity Analysis

If liquefaction-induced ground deformations and ground settlements are tolerable, the remaining possible liquefaction-induced hazard to bridges is loss of foundation bearing strength. Loss of bearing strength could lead to penetration of shallow or deep foundations into the liquefied sediment or to lateral displacement or buckling of piles as a consequence of reduced lateral resistance in the liquefied soil layers.

5.5.1 Data Required

A standard bearing capacity analysis may be used to assess bearing capacity for shallow foundations, with residual strengths assigned to liquefiable layers. For calculation of axial load capacity for deep foundations, liquefiable layers are commonly assumed to have negligible strength. Lateral load resistance of deep foundations is usually estimated by assigning a multiplier ranging from 0.1 to 0.3, depending on relative density, to lateral load resistance calculated for nonliquefied layers.

5.5.2 Analysis and Classification

If the load capacity analyses indicate an adequate factor of safety (say 1.5 or greater) against each of the various modes of failure, the site may be classed as non-hazardous and immune to detrimental effects of liquefaction even though liquefaction of some subsurface layers may occur. At this juncture of the investigation, all of the possible detrimental effects of liquefaction have been considered and determined to be nondamaging to the Bridge. Conversely, if the analysis indicates a marginal factor of safety (less than 1.5), unacceptable foundation displacements may occur during an earthquake and further investigation should be recommended for the site to confirm the hazard.

SECTION 6
PRIORITIZATION OF SITES FOR FURTHER INVESTIGATION

Application of the screening procedures outlined in Sections 2 through 5 yields one of the following four possible outcomes (Figure 2-1): (1) confirmed high liquefaction hazard--very high priority for further analysis and possible mitigation; (2) confirmed liquefaction susceptibility but unknown ground failure potential--high priority for further investigation; (3) insufficient information to assess liquefaction susceptibility--prioritize for further investigation; or (4) confirmed low liquefaction hazard--low priority for further investigation. Based on these outcomes, the following procedures are recommended for setting priorities for further investigation or mitigation of liquefaction hazard.

6.1 Confirmed High Liquefaction Hazard

Sites with confirmed liquefaction and ground displacement hazard should be given very high priority for additional investigation and development of possible mitigative measures. Prioritization at this level should consider the following factors: A primary criterion should be the importance of the bridge. Essential bridges, as defined in Standard Specifications for Highway Bridges (AASHTO, 1992), should be given priority for further investigation and mitigation over other bridges. Bridges with higher traffic volumes generally would be given priority over bridges with lower traffic volumes. Older bridges or bridges with weaker or more brittle foundations and structural components should be given priority over stronger and more ductile structures. Bridges scheduled for major renovation or replacement might also be given high priority. These considerations are provided as general guidance; highway agencies should weigh these criteria along with local needs to set priorities for further investigations and hazard mitigation.

6.2 Confirmed Liquefaction Susceptibility, but Unknown Hazard

Sites with confirmed subsurface liquefiable sediment or sensitive clay layers, but with unknown ground failure hazard, should be given high priority for further investigation. The further site investigation would usually include CPT and SPT soundings and laboratory testing to provide sufficient site information to conduct the analyses listed in Section 5. Prioritization of sites for further investigation should proceed using the same general guidelines as suggested in Section 6.1, with the following additional guideline. Most liquefaction-induced bridge damage has occurred at river or other water crossings. Thus bridge sites involving water crossings should be given priority for further investigation over non-water crossings, such as viaducts and overpasses.

6.3 Insufficient Information to Assess Liquefaction Resistance or Strength-Loss Potential

Where insufficient information is available, additional site investigations will be required to fully evaluate liquefaction and ground failure hazards. These investigations usually include additional drilling, SPT or CPT, and laboratory testing to identify and delineate liquefiable layers of liquefiable or sensitive soils, and analyses to define ground failure and bridge damage potential. Geologic and groundwater-table criteria in Tables 3-1 and 3-3 may be used for this prioritization. Sites associated with water crossings should be given priority over sites at nonwater crossings. Sites with geologic conditions indicative of high liquefaction susceptibility should be given priority over sites assessed as having moderate or lesser susceptibility. The guidelines listed in Section 6.1 should also be considered in setting priorities for further hazard investigation.

6.4 Low Hazard and Low Priority for Further Investigation

Sites categorized a low hazard and low priority for further investigation need not be further analyzed or prioritized for further study, except for very critical structures where a high level of performance is mandated. Nevertheless, engineers should apply appropriate screening criteria for liquefaction hazard when new data is developed, such as for a new bridge or highway segment. Liquefiable sediment may exist beneath a small percentage of sites classed as low hazard due to unusual local geologic conditions or inaccurately reported site information. Thus evaluations might be made as new information becomes available, but further specific investigations for liquefaction hazard are not required.

SECTION 7
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ISSN 1520-295X